HYDROGEOLOGIC ASSESSMENT OF
YORK SANITATION COMPANY LIMITED
WASTE DISPOSAL SITE NO. 4

.

by

Ground Water Protection Unit
Hydrology and Monitoring Section
Water Resources Branch

G.M Hughes, Chief
November 15, 1982

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Background

The York Sanitation No.4 Landfill site is located in the Oak Ridges
Kame Moraine complex. The site is in an upland area which slopes
abruptly into a lowland immediately to the north and more gently
downslope towards the south. To the north of the site, the lowland
area is swampy, to the east it contains Musselman Lake, and to the
west forms the headwaters of the East Branch of the Holland River.

Two hydrogeologic units have been defined beneath the site, the
Perched Aquifer in the near-surface deposits and below that, the
Main Aquifer. The Perched Aquifer consists of relatively thin,
permeable sand units of limited lateral extent in a matrix of less
permeable glacial till. Ground-water flow in the Perched Aquifer
generally follows the surface topography. In the northern part of
the site, flow is toward the lowland to the north and northeast and
in the southern part of the site, flow is to the south. Although
the Perched Aquifer is used by some shallow domestic wells in the
vicinity, it is not considered to be a major water resource.

The Main Aquifer underlies the Perched Aquifer and, beneath most of
the site, it is separated from the Perched Aquifer by unsaturated
deposits of sand and silty sand. Recharge to the Main Aquifer is
from the Perched Aquifer and also most likely from the swampy
lowland to the north of the site. Ground-water flow in the Main
Aquifer beneath the site is from the north and the east toward the
south and the west. The Main Aquifer is a major ground-water
resource used by many domestic wells and by the Stouffville
municipal wells.

Waste disposal operations began at this site in 1962 and both liquid
industrial wastes and solid wastes were received until 1970. There
is little information available on the amount and composition of
wastes disposed of at the site during this period. Since 1970, the
site has received solid municipal and industrial wastes.

There are considerable ground-water quality data available that show
that contamination from waste disposal activities is present in both
the Perched Aquifer and the Main Aquifer and is moving with the
ground water in these aquifers.

Site Impact

1. Contaminants which are believed to be associated with the
    disposal operations are present in two private wells in the Main
    Aquifer to the south and west of the site. Except for
    parameters naturally elevated in this aquifer, as for example
    iron, the contaminants are well within the limits considered
    acceptable for domestic water consumption by the Ministry.

    Although there is some potential for contaminants from the site
    to reach the Stouffville municipal wells, there is no chemical
    evidence that contaminants have reached these wells. However,
    insufficient, reliable quantitative hydrogeologic data are
    available to assess any potential impact.

2. Ground-water flow in the Main Aquifer appears to be from
    Musselman Lake toward the site and therefore the potential for
    the contamination of domestic wells in the vicinity of Musselman
    Lake by contaminants from the disposal site is considered to be
    very small. Although there are some aspects of the
    hydrogeologic regime in the area that are still not fully
    understood, no mechanism can be visualized for moving
    contaminants from the site through the Main Aquifer to these
    wells.

3. Because of the hydrogeologic characteristics of the deposits in
    the Perched Aquifer, it is extremely unlikely that water
    entering this system on the site moves more than a few hundred
    feet away from the site boundaries before infiltrating downward
    into the Main Aquifer. Thus, the evidence is very strong that
    contaminants from the site cannot move to wells in the vicinity
    of Musselman Lake.

4 . Water quality data from the domestic wells west of the northwest part of the site indicate that some of these wells have contained or now contain low levels of some contaminants. For a variety of reasons this contamination cannot necessarily be attributed to the waste disposal operation. Although, it is possible to postulate a component of ground-water flow to the west from the northwestern part of the site, it is improbable that wells along Vandorf Sideroad would be impacted.

5. There is a strong possibility that any future, major development of water resources on adjacent properties to the south, to the west and perhaps to the east would be impacted by contaminants from the landfill. Cones of capture of future adjacent wells could extend beneath the site and thus some part of the well yield would consist of contaminated water originating from beneath the site.

6. There is ground-water movement through the Perched Aquifer to discharge into the lowland areas to the north of the site, and probably to the east of the site as well. Although, there are insufficient data to assess this ground-water flow quantitatively, there is a potential for contaminant migration into the surface waters in these areas. This potential would increase with the filling of the northern and northeastern parts of the sites.

7. It is considered, for a number of reasons, that the Main Aquifer beneath the northern part of the site would be comparatively more sensitive to contamination than the portions to the south.

Present State of Knowledge and Needs

This landfill is in a complex hydrogeologic environment and a detailed and comprehensive hydrogeologic assessment is thus very difficult. This is further complicated by the fact that records of waste disposal at the site are incomplete. Therefore, despite the large amount of effort expended in monitoring this site, its

hydrogeology is still not well enough understood to fully assess the present and potential impact of the site on the ground waters beneath adjacent properties. To accomplish this, additional information would be required in three broad areas. These are:

1) the present composition, distribution and concentrations of contaminants in the landfill and in the soils above the Main Aquifer,

2) the present composition, distribution and concentration of the contaminant plume, including its base, in the Main Aquifer and,

3) the amount of natural attenuation provided by the site and the amount of attenuation required to achieve an acceptable level of contaminant discharge from the site.

Without a clear understanding of the foregoing, it is not possible to properly address:

a) a full and meaningful interpretation of the present monitoring data,

b) the potential magnitude of offsite contamination from the existing landfill and from any expanded landfill,

c) the applicability of any proposed contingency plan,

d) a monitoring program to fully describe the contaminant plume and,

e) the maintenance and restoration activities which could be required in conjunction with or as a result of present and any future operations.

The resources and time that would be required to gather this information are not addressed in this assessment. These could be substantial.

1. INTRODUCTION

This report consists of an assessment of the hydrogeologic data
pertaining to the York Sanitation Company Waste Disposal Site No.4
in the Town of Whitchurch-Stouffville, and was prepared at the
request of the Environmental Approvals Branch of the Ministry of the
Environment to assist in their review of the site and as a reference
document for use at the Environmental Appeal Board Hearing.

Documents that were used for this assessment are listed in Appendix
A. References in the text refer to these documents by number as
listed in Appendix A. Other sources of data on geology,
hydrogeology and ground-water quality used in this assessment are
discussed in Appendix B. With some limited exceptions, only data
available as of January 1982 have been used in this assessment.

The body of this report addresses the assessment and interpretation
of the hydrogeologic data that have been gathered. In as much as
possible, it does not address the Applicant's proposal. Any
discussions directly related to the Applicant's proposal and
concerns for that proposal are presented in various Appendices.

A number of subjects related to the hydrogeology of the site have
not been addressed in this review. These are:

a) The recent investigations of the organic chemicals in nearby
    domestic and onsite wells and the results of the drilling
    conducted in 1981, to assess the mounding in the refuse and the
    quality of the leachate. These are ongoing investigations and
    are being handled by the Central Region.

b) The construction and design of the cover, the liner and the
    leachate collection system. This is being addressed by the
    Environmental Approvals Branch.

c) The resupply of water to domestic wells. This is being
    addressed by the Central Region.

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d) The design of the contingency purge well system. This has been addressed by other workers (40, 41).

e) The historical development of the ground-water monitoring at the site. This will be addressed by the Central Region.

The location of the site is shown on Figure 1. It is located in Lots 14 and 15, Conc.8, Township of Whitchurch, Regional Municipality of York. The present Certificate of Approval permits filling on 73 acres (29 ha) (28,v.2,p.74). An additional 52 acres (21 ha) remain to be filled but do not have a Certificate of Approval. These acreages, along with a buffer zone of 61 acres (24 ha), comprise the total site area of 186 acres (74 ha) (28,v.2,p.74).
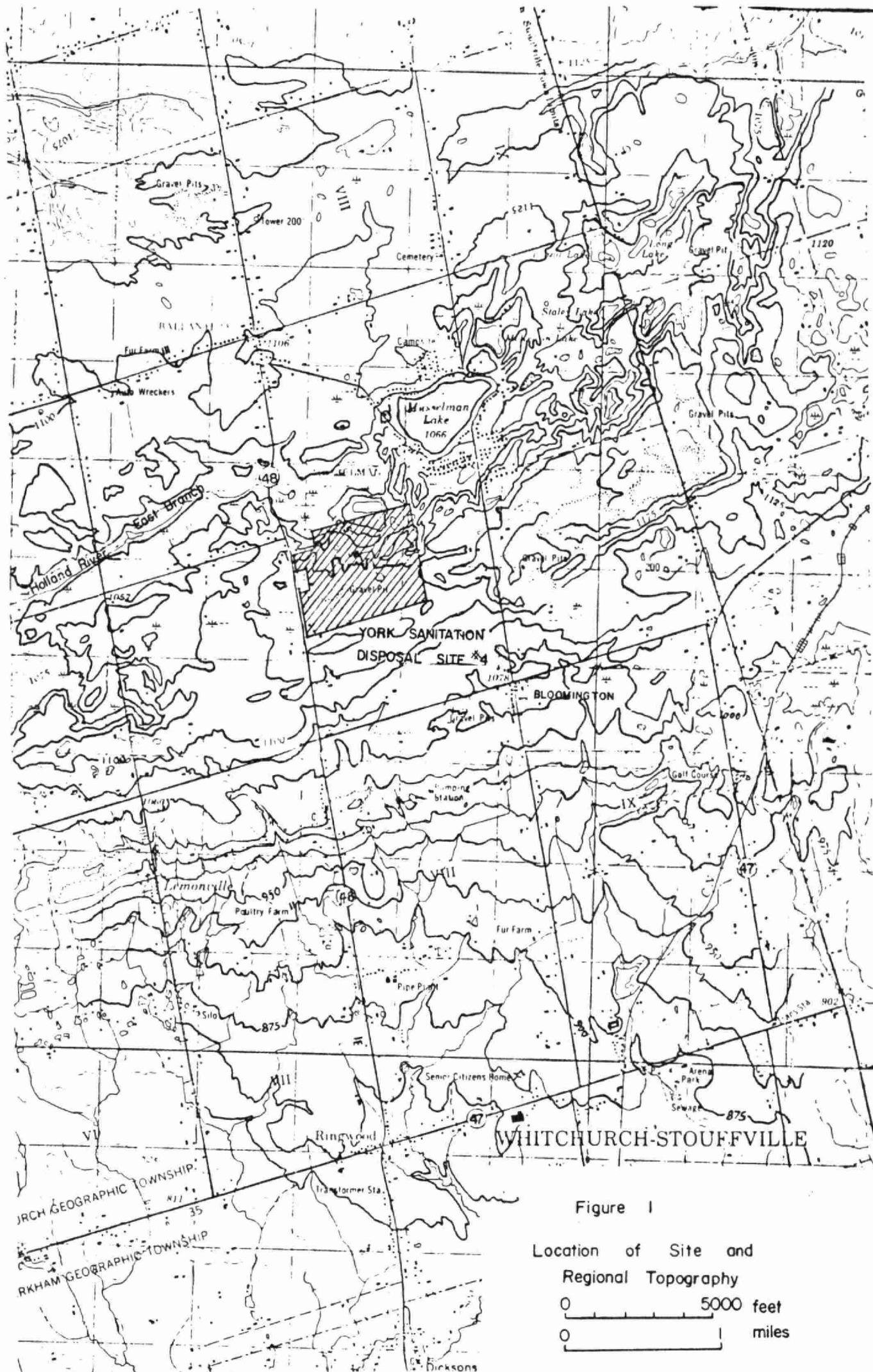
Figure 1

Location of Site and
Regional Topography

0   5000 feet

0   1 miles

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2. WASTE DISPOSAL AND CONTAMINANT RUNOFF

Waste disposal operations began at this site in approximately 1962 (2,p.2 and 6,p.3) and both liquid industrial wastes and solid wastes were received until 1970. During that period, solid wastes were co-disposed with the liquid wastes, (1,p.3), initially in the southwestern part of the site. Liquid waste lagoons were developed in closed depressions formed by glacial processes. These depressions are kettles and are discussed in more detail in Section 4.2.1 of this report. Figure 2 is a map of the site which indicates the areas of liquid and solid waste disposal in 1970. On December 27, 1969 (6,p.5) liquid waste disposal was temporarily discontinued and the practice was permanently terminated on June 15, 1970 (6,p.8).

On September 18, 1973, a plan was detailed by B. Beatty of Hydrology Consultants Limited. (6,App.C) for adding solid waste to lagoons or kettles of liquid waste, which still existed at the site. Subsequently, mainly during late 1973 and 1974, the lagoons were filled with solid waste and covered with earth (6,p.44, 29,p.1).

The present disposition of the solid wastes on the site is described in the progress reports submitted by the Applicant (10,p.11). Except for the access road, landfilling of solid wastes is more or less completed in the southwestern and south central parts of the site. Future landfilling is proposed for the eastern and northern parts of the site.

At the present time, only solid wastes are received at the site. According to the Applicant (28,v.4,p.469) the wastes include much solid industrial waste which produces a "weak" leachate with a relatively low concentration of chlorides, compared to that from domestic wastes.

There are little data available on the amounts and types of wastes received and the disposal practices during the 1960's. Some of the more pertinent information is presented below:

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LEGEND

↓↓	SWAMPY AREA
— —	OUTLINE of EXISTING WASTE FILLED AREAS
⁀⁀⁀⁀	OUTLINE of KETTLE
⊘	EXISTING WASTE LAGOON
✗	DIKE with DIRECTION of NATURAL DRAINAGE

N. M°LEAN

E. FOCKLER

HIGHWAY No. 48

WOODED AREA

W. BAKER

2

3

1

5

6

4

BAKER POND

K. HUTCHISON

M. BAKER

SCALE: 0   500   1000 ft.

TOWNSHIP OF WHITCHURCH

Investigation of Bremner Waste
Disposal Site

Map of Bremner Property Showing Loca-
tions of Boreholes and Waste Lagoons

Hydrology Consultants Limited

January 15, 1970   bwb

Figure 2

Modified from Reference #4

Figure 2   Location of Waste Lagoons and Landfill, 1970

1) A 1966 report by C.L. Young of the Ontario Water Resources
    Commission (1,p.2) notes that by July 1966 approximately 8 acres
    (3 ha) had been filled and covered and another 4 acres (2 ha)
    were being filled. It was estimated by the operator that
    approximately 70 yds^3 (54 m^3) of domestic and commercial
    refuse were received each day. Volumes on the order of 50 to 60
    thousand gallons (230 to 270 m^3) of sulphuric acid (3%) and
    calcium hydroxide were disposed of each month (1,p.3). These
    wastes, together with oil wastes were dumped on top of the solid
    waste and covered with earth.

    At that time (July 1966) it was estimated from available records
    that the "centre" area (probably the south-central part of the
    site) had received domestic garbage, commercial refuse, 105
    thousand gallons (477 m^3) of acid and 52 thousand gallons (236
    m^3) of waste oil (1,p.4). In addition the "rear" area
    (probably the area of Lagoon No.1) of the site had received
    several truck loads of liquid industrial waste in a pond with an
    area of about 60 yds^2 (50 m^2) (1,p.5).

2) A January 1970 report (4,p.4) by Hydrology Consultants Limited
    describes six areas containing liquid waste on the site (Figure
    2). Areas 1,2,3, 4 and 6 were disposal areas. These were
    kettle depressions, some of which had been "improved" by the
    construction of berms so that they could hold additional
    quantities of liquid waste. Liquid wastes exfiltrated more
    quickly from the Area 6 kettle, than from the other kettles
    (5,p.12) and thus Area 6 received a large volume of liquid
    waste. For the most part, Area 5 received runoff from Area No.4
    and was not a true disposal area (26,v.1,p.83).

    In the Hydrology Consultants Limited, September 1970 report
    (5,p.3) it was noted that "..... during the last eight years and
    to a greater degree during the last four years, several million
    gallons of liquid industrial waste from numerous sources have
    been dumped into natural closed depressions ...." on the
    site.

3) In testimony to the Environmental Hearing Board in 1974-75
(26,v.20,p.4426) the Applicant noted that the liquid industrial
wastes received at the site included oily wastes from service
stations, clean-up wastes from the food industry, hydrochloric
acid, sulphuric acid and sludges from water pollution control
plants.

In various documents describing this site, mention was made of
surface drainage of contaminated runoff into the following areas:

a) To the southwest of the site into low areas on both sides
of Highway 48, (1,p.4; 4,p.9; 28,v.3,p.227; 38,p.2)

b) To the north of the west side of the site into, and north
of, Lagoon 5 (28,v.13,p.227)

c) To the east of Lagoon 1 (6,p.18, 26 and 37, 28,v.3,p.226;
v.5,p.907). Liquid waste discharge from Lagoon 2 into
"water ponds" both inside and outside the eastern property
boundaries was also noted (6,p.26). This may be a
typographical error and perhaps should refer to Lagoon 1
instead.

There was also a small solid waste disposal site on the edge of the
upland near Highway 48 north of the northwestern part of the site
that was closed in 1960 (14), and the Applicant noted that solid
wastes are buried in the property south of the southwestern corner
of the site (38,p.3; 16,v.1,p.32).

3. TOPOGRAPHY AND DRAINAGE

3.1 Regional

Figure 1 is a section of the Markham and Newmarket 1:50,000
topographic maps showing the topography in the vicinity of the
site. The site is located on the top and north flank of a broad
regional east-west trending upland at an elevation of 342.9-350.5 m
(1125-1150 ft). This upland slopes rather abruptly downward into an
east-west trending depression to the north at an elevation of
320.0-327.7 m (1050-1075 ft). The eastern part of this depression
contains Musselman Lake. The depression immediately north of the
site contains a swampy area and an unnamed lake. This depression is
shown on the topographic map to be the source of the head waters of
the East Branch of the Holland River which flows from this
depression to the west. To the south of the topographic divide the
land slopes relatively gently downward to the south. A schematic
north-south cross-section through the site showing this topography
is presented as Figure 3.

3.2 Site

A series of contour maps have been prepared by the Applicant for
various submissions (6,10,11,16). These maps are on an air-photo
base with .76m (2.5 ft) and 1.5 m (5 ft) contour intervals at a
scale of 1:1200. The southern part of the site is an upland with an
elevation of about 344.4 m (1130 ft) above sea level. To the north
the site slopes into a swampy area which is at an elevation of about
320.0 m (1050 ft). Before landfilling, the topography of the upland
surface was irregular particularly in the north, and closed
depressions (kettles) were present. These closed depressions acted
as surface water collection points. This resulted in a minimal
amount of surface runoff being discharged overland prior to waste
disposal (6,p.18). It was noted (6,p.26) that before the waste
disposal operation began "... the basins (kettles) would rarely
overflow whereby a continuous flow of runoff through the site would
occur".

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4. GEOLOGY AND HYDROGEOLOGY

4.1 Introduction

The geology and the hydrogeology of this site are addressed with
respect to both regional and site characteristics. In addition,
geologic and hydrogeologic units have been defined as shown on Table
1 and these units are used throughout this document.

The discussion of the geology is presented to provide a framework
that has some meaning with respect to the genesis of the geologic
materials and their related physical characteristics.

The discussion of the hydrogeology addresses ground-water flow and,
as ground water is the means by which contaminants are transported,
serves as the basis for Section 5 of the report dealing with
contaminant hydrogeology.

4.2 Geology

4.2.1   Regional Geology

The area is a part of the Oak Ridges kame moraine. In the vicinity
of the disposal site, the Oak Ridges Moraine is overlain by the
Palgrave Moraine composed of Halton Till (36). Beneath the kame
moraine deposits is a thick series of glacial and interglacial
deposits. These overlie the bedrock which is at a depth greater
than 168 m (550 ft).

The term "till" refers to geologic materials deposited directly from
glacial ice. Typically, tills are unsorted mixtures of sand, silt,
clay and stones left by a glacier.

The Oak Ridges kame moraine, is a geomorphic feature formed by the
continental glaciers. Kame moraines are characterized by irregular
topography. There are often closed depressions present called
kettles, which are formed by the melting of large blocks of ice

UPLAND ·········· LOWLAND

Hydrogeologic Unit	Geologic Unit		Geologic Unit	Hydrogelogic Unit
Perched Aquifer(s)	Upper Unit			
	Lower Unit		Upper Unit	
Main Aquifer	Clayey Deposit	?	Lower Unit	Main Aquifer

◻ mostly sand, gravel and silt

◻ mostly clay and till
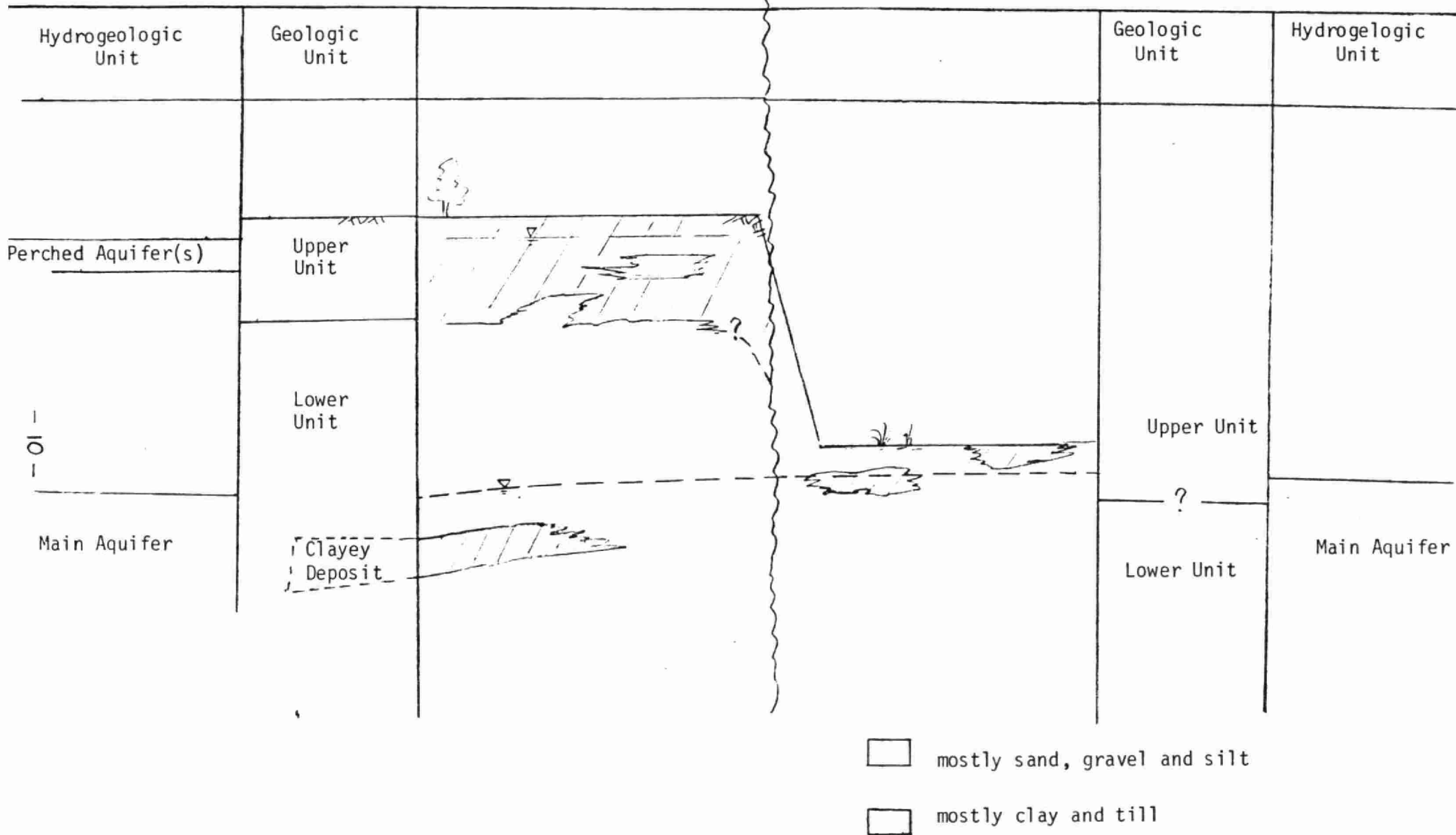
Table 1   Geologic and Hydrogeologic Units

buried in the debris left by the retreating glacier. The geologic materials in kame moraines generally have a wide range of texture. For the most part these materials are ice contact in origin, and consist of gravel, sands, silts, and clays deposited by glacial melt-waters as well as deposits of glacial till. Individual beds or units in kame moraines can seldom be correlated with confidence over distances greater than a few hundred metres.

More complete discussions of the glacial deposits in this area are presented in published reports and maps (35,36).

4.2.2   Site Geology

4.2.2.1 Geologic Units - The geologic materials at the site which are pertinent to this assessment, have, for the purposes of this report, been divided into two broad units: the Upper Unit and the Lower Unit. These are illustrated on Table 1 and on Figure 3, a schematic cross-section through the site.

4.2.2.2 The Upper Unit - In the southern part of the site the Upper Unit is generally 9 to 15 m (30 to 50 ft) thick and consists primarily of sandy, silty and clayey tills with some beds of silt and fine sand. To the north the Upper Unit contains less till and a greater proportion of sand and silt. As a result, the Upper Unit becomes poorly defined and is identified in the lowland to the north only because the upper part of the geologic sequence contains a greater amount of fine textured geologic material in some of the borings in this area. The Upper Unit in the lowland is not considered to be depositionally equivalent to the Upper Unit in the upland.

4.2.2.3 The Lower Unit - This Unit consists primarily of sand, gravel and silt deposits. In the south-central part of the site this Unit is more than 40.5 m (133 ft) thick (OW7-73). According to U. Sibul, (personal communication), based on regional data the deposits of the Lower Unit could be on the order of 45 to 60 m (150 to 200 ft) thick beneath the site, and are probably underlain by a glacial till.

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The thinnest sequence of sand and gravel in the Lower Unit was
logged in borehole OW1-81. However the description of the deposits
found in this borehole is not consistent with that of the materials
in nearby boreholes where there is considerably more sand.
Geophysical logging supports the description of the materials logged
in the other boreholes and thus the 3.4 m (11 ft.) thickness of sand
shown in the OW1-81 log is not considered to be typical.

In the southwestern part of the site a fine textured deposit
containing clay is present in the Lower Unit. This clayey deposit
was not found in other parts of the site. It is significant to the
assessment of the hydrogeology of the Main Aquifer beneath the site
and is discussed further in Section 4.3.4.4.

In the southern part of the site the elevation of the interface
between the Lower and Upper Units is approximately 335.3 m
(1100 ft). To the north this boundary cannot be readily
distinquished.

4.2.2.4 Genesis of Deposits - Based on a report by Sibul et al
(35,p.15), the Lower Unit is part of a large deposit comprising the
core of the Oak Ridges Moraine. It was deposited by melt waters,
between northward and southward retreating ice lobes, during the
late stages of glaciation.

A possible genesis for the materials found in the Upper Unit at this
site might be as follows. Assume the last of the glacial ice to
melt was in the lowland area to the north of the site. The mixture
of deposits in the lowland in the northern part of the site might be
explained as relatively thin deposits of glacial debris slumping off
the sides of the residual ice. The more consistent deposits in the
Upper Unit in the uplands in the southern part of the site, might be
explained as earlier deposits, predominantly of till, deposited
directly by the continental ice sheet during a re-advance of this
ice.

4.2.2.5 <u>Relationship to Applicant's Submission</u> - The geologic subdivisions used in this report are related to those used by the Applicant as follows:

The Upper Unit is equivalent to the Applicant's "Confining Formation" and the Lower Unit to the "Aquifer Formation" as shown on the cross-sections on Figures P3 to P6 inclusive in the Applicant's submission (16,v.1). On these cross-sections the boundary between the Confining Formation and the Aquifer Formation is generally drawn at the base of the deepest deposit that contains an appreciable amount of fine textured material, above the Main Aquifer.

The Applicant assessment does not suggest that there is any horizontal continuity in the individual units in the Confining Formation. The Confining Formation is not a formation in the geological sense but is only considered for its content of fine textured material.

4.3 HYDROGEOLOGY

4.3.1   Introduction

Two major hydrogeologic units are present at the site; a Perched
Aquifer (or perhaps more accurately, Aquifers) in the Upper Unit and
a Main Aquifer in the Lower Unit. These Units are identified on
Table 1.

There is some uncertainty regarding the aquifers used by the wells
on the W.E. Fockler and W. Mehaffey properties. These are discussed
in Appendix B.

4.3.2   The Perched Aquifer

4.3.2.1 General Comments - The Perched Aquifer(s) contains ground
water that has infiltrated into the Upper Unit. This water is in
transit through the Upper Unit, both downwards into the Lower Unit
to recharge the Main Aquifer and laterally to discharge into the
swampy area to the north of the site.

The Perched Aquifer is probably used to supply water to one of the
W.E. Fockler wells in the vicinity of the site and is also used by
other domestic wells in the general area. It is not considered to
be a major water resource.

Thin beds of sand and silt are the conductive units of the Perched
Aquifer. These units have hydraulic conductivities that are orders
of magnitude greater than those of the associated glacial till and,
within the glacial till, could have a major influence on the
movement of ground water in the Perched Aquifer. It is also
possible that there are fractures in the fine textured deposits
associated with the Perched Aquifer and, if this is the case, these
fractures would also affect ground-water flow. Based on examination
of the soils boring data in this and other assessments (6,p.24) and
considering the characteristics of other deposits with similar
genesis, it is not considered that individual conductive units or

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"beds" in the Perched Aquifer are continuous across the site and thus it is expected that local ground-water flow within the Perched Aquifer would be complex.

Figure 4 is a map, based on the soils boring data, showing where, it is judged that the geologic materials in the Upper Unit or the near surface deposits allow relatively fast and direct access to the Main Aquifer i.e., would not provide a significant "barrier" to the downward migration of water into the Main Aquifer. Figure 4 was prepared to provide some explanation for the ground-water mounding in the Main Aquifer discussed in Section 4.3.4.1. It does not address the attenuation capacity of various parts of the site.

It was considered in the construction of Figure 4, that the presence of a unit with relatively low hydraulic conductivity, at least 3.0 m (10 ft) thick within the upper 6.1 m (20 ft) of the surficial deposits would constitute such a "barrier". It was assumed that thiner deposits less than $\overset{3}{\cancel{6.1}}$ m ($\overset{10}{\cancel{20}}$ ft.) thick of materials with relatively low hydraulic conductivity would not be laterally continuous to the degree that they would provide an effective barrier.

A considerable amount of professional judgement was used in constructing this map and the criteria stated above were not rigorously followed at all times.

In the southern part of the site, borings in the Upper Unit commonly were reported to encounter ground water that is a part of the Perched Aquifer(s). If these borings intersected the top of the Lower Unit or perhaps silty or sandy units in the bottom part of the Upper Unit connected to the Lower Unit, the water in these borings drained into the sands of the Lower Unit and to the Main Aquifer. The water levels found during drilling were used in preparing a map of the water table discussed in Section 4.3.2.5.

Perching is less easily accomplished in the northern part of the site where there appears to be less fine-textured material in the

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Upper Unit (Figure 4) and thus the vertical hydraulic conductivity is higher. In this part of the site it is not meaningful, for the purposes of this assessment, to distinguish the Perched Aquifer(s). This relationship is also shown on Figure 8, a north-south cross-section through the site and is discussed further in Section 4.3.4.1.

4.3.2.2 Recharge and Infiltration - There has been considerable discussion in various submissions and testimony of the amount of infiltration that moves through the Perched Aquifer to recharge the underlying Main Aquifer. The discussions were generally related to one of the following issues:

a)   The amount of dilution that would be available in the Main Aquifer to reduce the concentration of contaminants;

b)   The amount of leachate that would be produced; and,

c)   The reduction in the amount of water available to wells completed in the Main Aquifer, as a result of the landfilling operation.

Our comments on the first two of the above issues follow:

1.   None of the estimates that have been made of the amount of infiltration can be confirmed as being the most reliable or representative. (The data from the lysimeters on the site was not assessed in this review). Therefore it is necessary to use conservative estimate of infiltration in dilution and leachate production.

2.   The difference between the maximum and minimum estimates of infiltration is not large compared to the uncertainties involved in the other parameters used in the estimates of dilution and leachate production.

3. Calculations incorporating the values for infiltration lead only to an indirect assessment of the impact of the site. As discussed later in this document direct measurements of contaminant levels provide a more reliable assessment.

Because of the foregoing it is not believed that further quantification of infiltration represents a major issue at this stage of the assessment of the impact of this site.

A number of estimates of infiltration have been presented by various workers as follows:

1) A figure for infiltration of 225,000 igpd/mi^2 (equivalent to 5.7 in/y (144 mm/y)) was suggested (2,p.7) as an infiltration rate for the general area in an early study and this figure was used by the Applicant in computing a water balance (19,p.20).

2) Based on a 1974 report, the Applicant proposed a figure of 3 in/y (76 mm/y) (13,p.7) for infiltration through those parts of the site covered by surficial tills.

3) The Applicant stated (6,p.34) that after the site is completed the rate of leachate production will be 0.2 igpm/acre (183,000 igpd/mi^2), equivalent to 4.6 in/y (117 mm/y).

4) V. Dixon, in an independent assessment (8,p.4), suggested that the recharge rate would vary throughout the general area; from about 10% or less of the precipitation of 32 in/y (812 mm/y) where the confining layer is 50 ft (15 m) thick or more, to more than 30% where it is absent.

5) A relatively high estimate of infiltration can be based on the assumption that there is a negligible amount of runoff from the site, and that the mean annual water surplus (37,p.54) of 11 in (279 mm) would therefore represent infiltration on the site. This assumption is supported by an early report on the site, in which it was noted that much of the surface drainage on the site

was into the kettles (6,p.18) and although these kettles occasionally overflowed, there was a minimal amount of surface runoff from the site. Therefore most of the time, the capacity of the surficial materials was adequate to accept all of the normal precipitation (water surplus).

With reference to the third issue, in discussion of the depletion of the Main Aquifer as a water resource, the Applicant expected (28,v.4,p.383) that there would be no significant difference in recharge to the Main Aquifer as a result of landfilling for the following reasons:

a) Because of the low hydraulic conductivity of the surficial till deposits (point 2 above), before landfilling there would have been significantly less than 144 mm (5.7 in) of infiltration in a year on the site as a whole,

b) The completed landfill will allow two to four inches (51 - 102 mm) of infiltration per year,

c) There is no significant difference between these rates in this context.

It is felt that this issue of the reduction in recharge to the Main Aquifer is not important to the assessment of the hydrogeology of this site in the context of depleting a major water resource. The area of the site is relatively small compared to the total area providing recharge to the Main Aquifer, and even the complete elimination of recharge on the site would likely have little impact on the Aquifer as a whole.

4.3.2.3 Hydraulic Loading and Seepage - The purpose of some of the early hydrogeologic work on the site (4, 5) was to investigate the vertical and horizontal distribution of contamination in the vicinity of the kettles that received the liquid industrial waste. The work was an attempt to define the extent of this fluid in the Perched Aquifer. In these studies, fluid from the lagoons was detected in the Perched Aquifer and in the Main Aquifer.

When the lagoons containing liquid industrial waste were filled with refuse in 1973 and 1974, the fluid levels in the lagoons rose and presumably in the Perched Aquifer as well, as a result springs developed on the sides of the filled areas. According to the Applicant (28,v.20,p.3512), the seepage from these buried lagoons in 1975 was as follows:

a)   Seepage was continuous around Lagoon No.5 with flow to the north,
b)   Seepage was partially continuous from Lagoon No.2,
c)   There were two major seeps from Lagoon No.1, one to the north and one to the east,

In 1978, there was continuous seepage from Lagoon No.4 along the access road (28,v.20,,p.3513). For the most part these seepages have now been controlled and according to the Applicant, (28,v.20,p.3513) there were only three small seepage points on the site (1981).

Some drilling was done in 1981 to identify fluid levels in the lagoon areas. No fluid was encountered during the drilling except in the vicinity of Lagoon 5. The testimony at the 1981 Environmental Assessment Board Hearing is not clear. However, the absence of fluid appears to have been attributed to the borings having missed the deepest parts of lagoons and/or to a decline in fluid levels (28, p.3690-97) because of infiltration into the Main Aquifer. This aspect of the site investigation is continuing and being handled by the Central Region.

4.3.2.4 Hydraulic Conductivity - Work conducted to determine the hydraulic conductivity of the materials in the Perched Aquifer has yielded a range of values from 10^{-2} to 10^{-7} cm/s (10^2 to 10^{-3} igpd/ft^2). This wide range is not unexpected in these types of deposits. The higher values would represent sands, the lower, unfractured glacial tills. A major factor in dealing with ground-water flow in the Perched Aquifer is the continuity (extent and interconnection) of the various sand or silt units that have

relatively high hydraulic conductivity. This is because the ability
of a unit to conduct ground water is directly proportional to its
hydraulic conductivity and thus a one centimeter thick sand unit (or
fracture) with a hydraulic conductivity of 10^{-2} cm/s (10^2
igpd/ft^2) would, other factors being equal, conduct as much water
as a 1000 m (3280 ft) thick till unit with a hydraulic conductivity
of 10^{-7} cm/s (10^{-3} igpd/ft^2 ft). Therefore, where relatively
high conductivity units are interconnected they could control
ground-water flow.

Detailed work on the liquid waste lagoons (6,p.24) and examination
of excavations on the site (28,v.3,p.283) indicates that units with
relatively high hydraulic conductivity are not interconnected. On
the other hand, the absence of surface run-off and the loss of
liquid waste in the lagoon areas by exfiltration to the Main Aquifer
suggest the presence of some level of interconnection and an
"effective" hydraulic conductivity in the Perched Aquifer
substantially higher than 10^{-7} cm/s (10^{-3} igpd/ft^2).

For the purposes of this assessment it has been concluded that the
geologic materials in the Perched Aquifer (Upper Unit) are
sufficiently conductive to allow most if not all of the leachate
that has been, or is likely to be produced at the site, to move
downward into the ground rather than to run off as surface flow.
More specific data would be required to develop a detailed
understanding of ground-water flow in the Perched Aquifer.

Some of the data that has been accumulated on the hydraulic
conductivity of the materials in the Perched Aquifer and associated
deposits is outlined below.

1. Based on somewhat qualitative judgements (5,p.6) of water level
    declines during falling head testing (input testing) on
    monitoring wells (Appendix B), the hydraulic conductivity of the
    fine textured materials in the Upper Unit was estimated to be on
    the order of 10^{-6} cm/s (10^{-2} igpd/ft^2).

2. Based on grain-size distribution curves and falling head
   permeameter tests, (12,p.3, App.A and 13,p.5,Table 1) hydraulic
   conductivities of various deposits associated with the Perched
   Aquifer range from 10^{-2} to 10^{-7} cm/s (10^2 to 10^{-3}
   igpd/ft^2) depending on the texture of the particular material
   tested. Using these procedures the tills in the southeastern
   part of the site were assigned a hydraulic conductivity on the
   order of 10^{-7} cm/s (10^{-3} igpd/ft^2), (13,p.4) by the
   Applicant.

3. Based on the rate of lateral migration of contaminants from a
   disposal lagoon of 2.5 to 4 ft (0.8 to 1.2 m/y) (4,p.10) it was
   estimated that the hydraulic conductivity of the till would be
   on the order of 10^{-5} to 10^{-6} cm/s (10^{-1} to 10^{-2}
   igpd/ft^2) (6,p.6). Gillham (30,p.3) in reassessing these
   data, estimated the hydraulic conductivity to be 1.1×10^{-4}
   cm/s (2.82 igpd/ft^2) to 2.2×10^{-4} cm/s (5.64 igpd/ft^2).

4.3.2.5 Ground Water Flow Direction - The generalized water table
in the Perched Aquifer is contoured on Figure 5. The slope of the
water table generally follows the slope of the land surface; that
is, to the north towards the swamp with some indication of southerly
flow in the southwestern part of the site. It is possible that
there is some spring discharge, or seepage, from the Perched Aquifer
on the slopes south of the swamp. This has not been confirmed by
field inspection.

It should be noted that Figure 5 probably does not represent the
water table in a continuous "conductive" unit or deposit, because,
as stated previously, (Section 4.3.2.1) the individual "aquifer"
units in the Perched Aquifer are probably not well interconnected.

4.3.3   The Main Aquifer - Regional

4.3.3.1 General Comments - The bottom part of the Lower Unit in the
upland beneath the southern part of the site and those more
conductive parts of the surficial deposits in the lowlands in the

north of the site that are saturated, are for the purpose of this
report designated as the Main Aquifer (Table 1). Although
correlations of individual units are uncertain in kame moraine
deposits, this large aquifer appears to extend for at least a few
thousand feet or hundreds of meters to the south, west and east of
the site and appears to be continuous with the aquifers used by the
Stouffville Municipal Wells (6,p.20) as well as the deep domestic
wells near Musselman Lake. Although the correlation becomes less
reliable to the north, this aquifer may also serve deeper wells in
the vicinity of Ballantrae. The Main Aquifer is a major regional
water resource.

4.3.3.2 <u>Ground-Water Flow Direction</u> - Figure 6 shows a
generalization of the regional piezometric surface of the Main
Aquifer. Based on Figure 6, the regional ground-water flow
direction in the Main Aquifer is from the north and east toward the
south and west. The location of the ground-water divide, north of
the site is not certain. The regional mapping done in this
assesment suggests that this divide extends east and west in the
vicinity of Ballantrae. The Applicants have indicated
(16,v.1,p.15,map 2) that the divide is concentric around the area of
Musselman Lake. It is also possible, based on topography and
general hydrogeologic considerations, that the divide could be
located beneath the swampy depression to the north of the site.

A degree of uncertainty is not unusual in this type of work and the
location of the divide is not critical to the present assessment of
the site. This is explained more fully in Appendix C. Figure 6 is
based on the assessment of the drilling records of domestic and
municipal wells from the Ministry's files. Maps produced from these
types of records are much less reliable than maps based on data from
wells specifically constructed to gather hydrogeologic data. The
construction and interpretation of regional maps is discussed in
Appendix B.

There is uncertainty about hydrogeologic conditions beneath the
swampy area and Musselman Lake. In those areas, the hydraulic

gradient appears, from the regional data, to be downward but it is not certain if there is perching of the surface waters with the presence of underlying unsaturated deposits or if saturated conditions are continuous with depth. There is additional discussion of regional ground-water flow in Appendix C.

4.3.3.3 Effect of Municipal Wells and Impact of Ground Water Development on Regional Ground Water Flow - Figure 6, and the regional maps prepared by other workers (6,Fig.A-3, and 8, Dwg.B-47522) do not show any influence of withdrawals from the Main Aquifer by the Stouffville Municipal Wells.

The reasons for this include:

a)   the accuracy of the regional water level data is probably on the order of $\pm$ 6.9 m, (20 ft),

b)   the maximum drawdown possible at the municipal wells is in the order of 12 m (40 ft),

c)   only relatively large features in gradient would be apparent at the scale of 1:25,000 and the contour interval of 25 feet, used in Figure 6.

The amount of potential impact on ground-water levels and ground-water flow directions of the present and future development of the ground water resource in the Main Aquifer has not been addressed in detail in this assessment. Although further development in the vicinity of the site may be anticipated, its timing and location cannot be predicted. Based on data collected during the testing of the Stouffville Municipal Wells, as discussed below, it can be anticipated that any major water taking from the Main Aquifer would affect ground-water levels for a distance of at least a few thousand feet or hundreds of meters. It must be assumed therefore that ground-water flow directions in the Main Aquifer beneath the site will be modified by future development of this resource.

Some of the data presented addressing the radius of interference caused by ground-water extraction is as follows:

1) A theoretical assessment of the Stouffville Municipal Wells, located approximately 2286 m (7500 ft) south of the site, indicates that the cone of interference from these wells could extend for a radius of 1829 m (6000 ft) (8,p.4).

2) Our assessment of the data from the pumping tests of the Stouffville Municipal Wells (Appendix D) indicates that, depending on the methods used to assess the data, estimates of the distance at which zero interference could take place can vary by a factor of 10 times. However, in our judgement a figure of 1829 m (6000 ft) is not unreasonable. It is considered to represent a theoretical maximum value.

3) It was estimated that the cone of depression of the production well completed in 1966 may have extended to a radius of 610 m (2000 ft) when the pumping level in the well was about 4.6 m (15 ft) below the top of the screen (2,p.5).

4.3.3.4 Hydraulic Gradient and Conductivity - Regional horizontal hydraulic gradients in the Main Aquifer, based on Figure 6, are in the range of 0.005 to 0.05. No attempt was made to calculate any values for the regional vertical gradient from the off-site wells. However, as a qualitative judgement, it appears from the cross-sections (not presented) that in upland areas the deeper domestic wells have deeper water levels. Thus it is expected, as is normal in ground-water flow systems, that the vertical gradient in the Main Aquifer is downward beneath the upland areas.

Our assessment of the transmissivity of the Main Aquifer determined from the pumping data from the Stouffville Municipal Wells gives a range from 200 to 700 m^2/d (13,400 igpd/ft to 46,000 igpd/ft). These values are compatible with values of 300-600 m^2/d (20,000 igpd/ft to 40,000 igpd/ft) provided in other assessments (31,32,33,8,p.2).

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Considering an aquifer thickness of between 10 and 19 m (32-64 ft.) this yields a hydraulic conductivity for this aquifer of 8×10^{-3} cm/s (216 igpd/ft^2) to 6×10^{-2} cm/s (1400 igpd/ft^2). These values are discussed further and presented in Table 2 in Appendix D.

Estimates of the velocity of the ground water in the Main Aquifer range from 265 (16,p.19) to 1000 (5,p.11) ft/y (81-305 m/y). Our assessment of the data generally agrees with this range of values.

4.3.3.5 <u>Ground Water Recharge and Discharge</u> - In the upland areas, recharge to the Main Aquifer is directly from precipitation or through the local Perched Aquifer in the near-surface deposits. This has been discussed in Section 4.3.2.2. Additional regional recharge is received from the lowland areas such as those containing Musselman Lake and the swamps to the north of the site (Appendix C). In humid climates such as Ontario's, lowland areas generally are zones of ground-water discharge rather than recharge and hence, this recharge from a lowland is somewhat unusual. It is expected that to the west of Highway 48, where the East Branch of the Holland River becomes a permanent stream, ground-water discharges into that stream to provide baseflow and a more usual hydrogeologic regime will be present.

The most likely locations for regional natural discharge from the Main Aquifer would be:

a.   the flowing wells in the vicinity of Lemonville approximately 3.2 km (2 mi) to the south and west of the site,

b.   the streams approximately 1.6 km (1 mi) to the south of the site, which originate at an elevation of approximately 305 m (1000 ft) AMSL.

4.3.4   <u>The Main Aquifer - Site</u>

4.3.4.1 <u>Ground Water Flow Direction and Mounding</u> - Figure 7 presents the water-table surface of the Main Aquifer beneath the site. Figure 8 is a north-south cross-section showing the sequence

of deposits and the position of the water table. This cross-section
illustrates the relation between the Perched and Main Aquifers in
the northern part of the site as addressed in Section 4.3.2.1.
Ground-water flow in the upper part of the Main Aquifer is from a
ground-water mound in the northeastern part of the site towards the
south, west and east boundaries of the site. There is also flow
into the site across the western part of the northern boundary.

The location of the ground-water mound corresponds with a southerly
extension of the lowland to the north of the site. According to F.
Rovers (28,v.20,p.3470) almost one quarter of the site drains into
this low lying area around OW1-73. There is also an increase in the
hydraulic conductivity of the deposits in the Upper Unit in this
area as is shown on Figure 4 and discussed in Section 4.3.2.1. It
is believed that this combination of features allows more direct
recharge (i.e., without perching) to the Main Aquifer. Where this
direct recharge occurs, there is no Perched Aquifer. Greater
recharge in the northern part of the site was also suggested by the
Applicant (6,p.25) and the ground-water mound in the vicinity of
OW1-73 was recognized. This mound and the water-level elevations in
wells in the Main Aquifer are shown in Figures 7 and 8.

The presence of the ground-water mound is largely defined by data
from wells OW4-73, OW1-73 and OW2-78. For reasons associated with
the depth, the type of material in which these wells are completed,
and the rate at which they "recover" after sampling, (Appendix B),
there is some uncertainty as to whether the water levels in OW4-73,
and to a lesser extent OW1-73 and OW2-78, properly reflect
conditions in the Main Aquifer. Because of the difficulties in
interpretation the water-level data from these wells are plotted on
the water-table maps of both the Perched Aquifer (Figure 5) and the
Main Aquifer (Figure 7). The wells are also shown on Cross-Section
A-A' (Figure 8). If they reflect conditions above, rather than in
the Main Aquifer, the mound in the Main Aquifer would be smaller
than shown.

This ground-water mound is an important hydrogeologic feature at this site because:

a)   it identifies the point of origin for a portion of the ground water moving beneath the site in the upper part of the Main Aquifer, and demonstrates that recharge to the Main Aquifer is greater in the northern part of the site;

b)   it will affect ground-water flow into the site from the north and east, at least through the upper part of the Main Aquifer, and thus affect any quantitative assessment of flow in the Main Aquifer.

The annual rise and fall of the mound in OW2-78 is on the order of 1.8 m (6 ft) each year, indicating that about 0.5 m (1.8 ft) of recharge infiltration into the Main Aquifer beneath this area each year. This is calculated by multiplying the annual change in mound height 1.8 m (6 ft) by a specific yield of 0.30 for the aquifer materials. An effective porosity (which is assumed to equal the specific yield) of 0.30 was used by the Applicant, (16,p.19). This figure probably represents the material in the Main Aquifer with the greatest specific yield and thus yields a high value for infiltration. (The total annual rainfall in this area is 0.8 m (37, p.40)).

The extent to which the mound causes flow out of the site to the east is not known. Water level data from domestic wells in the vicinity of Musselman Lake (Figure 6) and from wells on the site (Figure 7) indicate that, for part of the year, there must be a trough in the water-table surface between the mound and Musselman Lake. If the highest water levels on site in OW1-73, OW4-73 and OW2-78 are considered, this trough would be to the east of the boundary of the site. If the lowest levels are used, the mound would be absent or the trough would be within the boundaries of the site between OW4-80 and OW2-78. The water-level fluctuations in OW2-78 appear to be related to seasonal recharge events and thus it is possible that this trough migrates back and forth across the site

boundary seasonally. A similar conclusion was reached by the Applicant (28,v.20,p.3477,3480).

There is not sufficient hydrogeologic information on the Main Aquifer to estimate how deep into the Main Aquifer the influence of this mound extends.

It should be noted that it is proposed to install a cover on the entire site to minimize infiltration and an earth liner with a hydraulic conductivity of 10^{-6} or 10^{-7} cm/s (10^{-2} or 10^{-3} igpd/ft^2) is to be placed over the part of the north central part of the site below an elevation of 328.3 m (1077 ft) AMSL. The cover and the liner should reduce and distribute the infiltration into the Main Aquifer, and consequently should reduce both the height of the mound and the amount of ground water that moves to the east. The amount of the impact of this cover and liner will depend both on the effectiveness in reducing infiltration and its coverage of those areas in which infiltration is greatest. This aspect of the Applicant's proposal is to be addressed by the Approvals Branch.

As noted previously, our interpretation of the regional water levels in wells completed in the Main Aquifer (Figure 6) indicates that ground-water levels in the vicinity of Musselman Lake are higher than those along the eastern border of the site. Thus, the flow of ground water (or contaminated ground water), could not be from the site to Musselman Lake through the Main Aquifer. As discussed in Section 4.3.3.2 and in Appendix B there are deficiencies in the use of water-level data from domestic wells and there are some aspects of ground-water flow in this area that are not clearly understood (Appendix C). However the evidence is very strong that ground water cannot move from the site to Musselman Lake, and no mechanism for such flow can be suggested.

4.3.4.2 Hydraulic Conductivity and Ground Water Velocity - The hydraulic conductivity of the Main Aquifer beneath the site has been estimated as follows:

a) 5×10^{-2} cm/s (1×10^{3} igpd/ft^2) based on the rate of movement of a contaminant peak. This is discussed in Section 5.4.4.6 of this report.

b) 10^{-3} cm/s (10^{2} igpd/ft^2) based on the rate of water-level declines in falling head, input tests on observation wells, (5,p.7) (Appendix B); and,

c) 1.1×10^{-2} cm/s (2.8×10^{2} igpd/ft^2) based on Hazen analyses of the grain-size distribution of the aquifer materials (12, App.A; 16;p.19).

As noted previously, hydraulic conductivity values extrapolated from data obtained from pumping tests at the Stouffville Municipal Wells are estimated to be on the order of 10^{-1} cm/s to 10^{-3} cm/s (10^{3} to 2 igpd/ft^2). (Table 2, Appendix D and 8,p.3). These are compatible with the other values for the hydraulic conductivity in the Main Aquifer beneath the site.

Using a hydraulic conductivity value of 5×10^{-2} cm/s (1×10^{3} igpd/ft^2), a representative horizontal gradient of 0.007 (Section 4.3.4.3) and an effective porosity of 0.30, (this is a reasonable estimate of the effective porosity of materials such as this) the average lateral linear velocity of the water in the Main Aquifer at the site would be 1×10^{-3} cm/s or approximately 366 m/y (1200 ft/y). This figure represents the velocity of ground-water flow through a "more conductive" section of the Main Aquifer.

The Applicant, using the same gradient and effective porosity (16, p.19), but using a hydraulic conductivity of 1.1×10^{-2} cm/s (2.8 x 10 igpd/ft^2 estimated the average linear velocity through the aquifer to be 265 ft/y (80.8 m/y).

Changing the values used for hydraulic gradient, hydraulic conductivity and effective porosity would change this velocity proportionately. Considering the uncertainty inherent in assigning

values to these aquifer parameters, a difference in velocity estimates of 5 times, as between the above two estimates, is quite acceptable.

The average pore water velocity in the unsaturated part of the Lower Unit was addressed by Gillham (30,p.7) who suggested that it would be on the order of 3.3 ft/y (1.0 m). This estimate assumes the volume of infiltrating water is 5.6 in (142 mm) in each year.

4.3.4.3 Hydraulic Gradient - The horizontal and vertical hydraulic gradients are required to determine the magnitude and direction or vector of the ground-water flow. The horizontal gradient of the water table of the Main Aquifer beneath the site, as shown on Figure 7, ranges from about 0.004 to 0.025. This is in general agreement with the Applicant who indicates a gradient on the order of 0.007 (16,p.18).

Vertical hydraulic gradients are addressed in Section 4.3.3.4 and Appendix E. There is a consistent downward vertical gradient of 0.06 between nested wells OW2-80 and OW3-80. However, the water levels in OW3-80 and OW4-80 suggest that the gradient decreases to 0.007 with depth. Additional water level data would be required to confirm this trend.

A small vertical gradient is indicated by the water levels in the two piezometer nests (OW1-75, OW2-75 and OW1-76, OW2-76) in the southwest portion of the site.

The measurements from the OW1-75, OW2-75 nest indicate that a downward gradient, with values in the order of 0.009 is present most of the time. Four out of 22 water level measurements obtained at this nest indicate an upward gradient.

The measurements from the OW1-76, OW2-76 nest indicate that an upward gradient, with values in the order of 0.004 exists for all but one of the 22 measurements.

We cannot explain the opposing vertical gradient in installations so close together in terms of the hydrogeology of this site. However, they are based on small measured differences in elevations, in the order of 25 to 50 mm (1 to 2 in), and thus may be related to an error in measurement.

4.3.4.4 The Thickness and Base of the Main Aquifer - The determination of the thickness and base of the Main Aquifer is important in the;

a) determination of the configuration of the ground-water flow system, the contaminant plume and the amount of dilution available and the

b) design of the contingency purge-well system.

The thickness of the Main Aquifer, near the southern boundary of the site, is more than 16 m (54 ft) at OW7-73. In the northeastern part of the site, OW4-80 penetrated approximately 38.1 m (125 ft) of the Main Aquifer. It is believed that neither of the wells penetrated to the base of the Main Aquifer. In OW1-81 in the southwestern corner of the site only 3.4 m (11 ft) of sand was logged in the Main Aquifer. It is felt that this aquifer thickness is questionable because the preponderance of data from other well records in the vicinity indicate a greater thickness of saturated sand to be present (See also footnote discussing Wade Irrigation Well in this Section). This sand thickness is also addressed in the underflow calculations in Appendix C and in Section 4.2.2.3.

The elevation of the base of the Main Aquifer is also uncertain. In the southwestern part of the site in borings OW1-75, OW1-76 and OW1-81, (Figure 14) the sands and gravels of the Main Aquifer are underlain by a fine textured deposit containing clay at elevations of 301, 294, and 295 m (987, 966 and 968 ft) respectively. The thickest section of this clayey deposit was found in Well OW1-81, which was completed after penetrating 15 m (50 ft) of "grey clay and pebbles". The clayey deposit was not encountered to the east in OW7-73, which was completed to an elevation of 297 m (974 ft) or to

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the west in OW1-74 which was completed to an elevation of 297 m (973 ft). This suggests that the base of the aquifer is deeper or the clay may be absent to the east and west[1].

Borehole OW4-80 in the northeastern part of the site penetrated to an elevation of 279 m (914 ft). A "grey silty fine sand with occasional gravel size pieces" was encountered in the bottom part of this boring. F. Rovers stated that this borehole is completed in "the aquitard" (28,v.21,p.3606) (A unit with relatively low hydraulic conductivity positioned between the Main Aquifer and some deeper aquifer). It is assumed that he believed this to be equivalent to the clayey deposit encountered in the southwest part of the site.

In summary, this "clayey" deposit, encountered in the southwest part of the site, is probably a local deposit and has limited lateral extent and the position of the base of the aquifer is uncertain.

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[1] According to the drillers log (well record 15835) the Wade irrigation well, located about 800 m (2600 ft) to the west of the site encountered clay at an elevation of 263 m (875 ft) AMSL. This well bottomed after penetrating 1 m (3 ft) of this clay. This well appears to have penetrated on the order of 27 m (90ft) of sand and gravel in the Main Aquifer (See also Appendix D).

5. CONTAMINANT HYDROGEOLOGY

5.1 General Comments

Contaminants introduced into the ground-water at this site can move with the ground water. In the Perched Aquifer, this would be primarily downward to the Main Aquifer and as discharge to the north into the swampy area. In the Main Aquifer, contaminant migration would be toward the south and west and possibly locally to the east of the site depending on the areas filled and the effect of the liner and cover on the ground-water mound, as discussed in Section 4.3.4.1.

As these contaminants move through the ground they are attenuated by various physical, chemical and biological processes. Of these processes, physical dilution is the simplest to quantify. Thus the first step in this assessment was to attempt to develop a coherent pattern or picture and a satisfactory explanation of the distribution of contaminants that are attenuated primarily by dilution. Such contaminants are termed conservative contaminants. At this site chloride is the most useful conservative contaminant. Hardness, specific conductance, sulphate and phenol were also considered because experience has shown that these parameters are often useful in assessing contaminant migration.

The pattern shown by the distribution of conservative contaminants in the subsurface beneath a site such as this should be generally compatible with the direction of ground-water flow and the time period and rate at which the contaminants were introduced into the ground-water flow system. It should also bear a relationship to the initial concentration of the contaminant and it should decrease in concentration in a manner compatible with the amount of dilution.

Contaminants move in a three dimensional plume with levels decreasing in all directions away from the central "core" of the plume. This three dimensional configuration must be considered in any assessment of the extent of contamination.

This Section of the report describes the attempts made to complete
this first step in the site assessment; i.e. to relate the
distribution of conservative contaminants to the hydrogeology of the
site. As it was not possible to accomplish this first step (i.e.,
to address, quantitatively, the attenuation of even the conservative
contaminants) to an acceptable level it was not possible to proceed
to discuss a second step.

Based on the literature, we can assume that there will be some
attenuation of conservative and non-conservative contaminants
produced at this site. Therefore in the following discussions, it
is assumed that leachate or contaminants migrating through the
subsurface are attenuated to some unknown degree and the fact of
this attenuation will not be repeated.

5.2 Sources and Characteristics of Contaminants

The history of waste disposal at this site was discussed in Section
2 of this report. The locations of the early disposal areas are
shown on Figure 2 and more recent disposal activities are described
in the Progress Reports (10,11) submitted by the Applicant.

The principle sources or the origins of contaminants at this site
would be the areas containing solid waste and the lagoons or kettles
which received liquid industrial waste. These sources occur
primarily in the southern part of the site.

In the early reports on the site it was noted that much of the
surface drainage on the site was into the kettles that were used as
liquid waste disposal lagoons (6,p.18). Thus, the liquid waste
disposal lagoons accepted not only the liquid industrial wastes, but
surface runoff and normal precipitation as well. Overflow and
surface runoff from these disposal areas was controlled, at least in
part by the construction of berms (4,p.4). Kettle No.6 (Figure 1,
Area 6) was noted to be particularly leaky (5,p.12 and
28,v.3,p.170). It was also reported (28,v.3,p.169) that there was
poor hydraulic connection between Kettles No.1, No.2 and No.3 and

the Main Aquifer and that the degree of connection at Kettle No.4
(5,p.12) was uncertain. During this stage of disposal each lagoon
would have developed its own contaminant plume.

It is expected that less localized contaminant plumes would develop
beneath the broader areas filled with solid wastes. However, it
would not be unreasonable for localized contaminant plumes to have
developed in these areas as well, where more conductive materials
are present in the Upper Unit.

Ground water can also be contaminated by activities other than
landfilling, as for example, by highway de-icing and by septic
systems. Both of these sources of contaminants are present in the
vicinity of the site and are a potential source of contamination for
the private wells.

The following information was found which is related to the quality
of the liquid industrial wastes and the contaminated water believed
to be very closely associated with the wastes at the site.

1)   Analyses of the chemical characteristics of five samples
     the liquid industrial wastes are from the site presented by
     the Ontario Water Resources Commission (3).

2)   Analyses of contaminated ground water from observation
     wells very close to the disposal lagoons were presented by
     Hydrology Consultants Limited. (4,p.7,Table 2 and 5, Table
     3).

3)   Analyses of ponds on the site which contained some leachate
     are presented by International Water Supply (8, Annex
     1,p.1).

4)   Analyses of seepage discharges from the sides of the
     landfill entered by the Applicant as exhibits at the 1981
     Environmental Assessment Board Hearing (28, Exhibit
     62,v.16,p.2984).

5) Samples of leachate were analyzed from wells installed by the Applicant in 1981 and '82. This work is part of the ongoing assessment program by the Ministry's Central Region and has not been considered in this review.

These data allow only a general understanding of the origins and composition of the wastes and the leachate. Needs for additional data are addressed more fully in the next Section of this report.

5.3 Contaminants Above the Main Aquifer

5.3.1   Introduction - An understanding of the contamination in the Upper Unit and in the deposits of the Lower Unit above the Main Aquifer is important to assess;

1) how much contamination remains in the waste, in the immediate vicinity of the waste and in the deposits above the Main Aquifer,

2) how much attenuation is provided by the deposits above the Main Aquifer, and, perhaps most important,

3) how much attenuation must be provided by the deposits in the Main Aquifer.

The data at the site are not adequate for us to properly address these questions.

The following discussion of contamination in the deposits above the Main Aquifer is addressed under three headings, as follows:

Historical quality of the ground water,
Downward migration of contaminants, and
Northward migration of contaminants.

5.3.2   Historical Quality of the Ground Water - Figure 9 shows the quality of the ground water in the Perched Aquifer. Data are not

sufficient for interpretation beyond observing that the contaminants are generally concentrated in the vicinity of Waste Disposal Lagoons 2 and 3. It should be noted that this is also the location of most of the monitoring wells and perhaps additional wells would identify other concentrations of contaminants.

Routine water quality analyses have been made of samples taken from the W.E. Fockler well, which is believed to use the Perched Aquifer. These analyses show that specific conductance values are slightly elevated in this water but this is not considered to be significant.

5.3.3   Downward Migration of Contaminants - Some of the early (1970) investigations at the site addressed migration of contaminants from the liquid industrial waste lagoons in the Perched Aquifer. These investigations traced contamination movement away from the lagoons in the Perched Aquifer at a rate of between 2.5 and 4 ft/y (1 m/y) (4,p.10).

At the 1981 Environmental Assessment Board Hearing (28,v.6,p.899), F. Rovers testified that contaminants would require eight to ten years to move through the Upper Unit to the Main Aquifer. He stated that leachate from Lagoon No.1 would migrate downward to the Main Aquifer at a rate of approximately 6 to 8 ft/y (2m/y) (28,v.22,p.3646).

He also stated that contaminants would move relatively quickly downward by fracture flow through the upper 15 ft (4.6 m) of the tills in the southwest part of the site. Continued downward flow would be at a velocity of 3 to 6 ft/y (1-2 m/y). He felt that there is more sand in the tills in the southeastern part of the site which would allow for higher downward velocities (28,v.22,p.3649).

As noted in Section 4.3.2.4, Gillham (30,p.7) suggested that pore water, would move downward through the unsaturated deposits above the Main Aquifer at an average velocity on the order of 3.3 ft/y (1.0 m/y). This rate would apply to conservative contaminants as well.

Excepting the early 1970 investigations, the estimates of contaminant migration presented above are largely based on indirect evidence and not on direct measurements of contaminant distribution and levels. Our opinion, also based largely on indirect evidence is that:

1)   As noted in Section 4.3.2.4 the hydraulic conductivity of the Upper Unit is probably sufficient to allow most if not all of the leachate produced at the site to infiltrate into the subsurface.

2)   In much of the southern part of the site where the deposits above the Main Aquifer are in the order of 30 m (100 ft) thick, contamination is probably still in transit, moving from the surface through the Upper and Lower Units toward the Main Aquifer.

3)   There was, and perhaps still is, rapid migration of contaminated water to the Main Aquifer through the more conductive areas or "windows" in the Upper Unit, such as Lagoon No.6.

5.3.4   Lateral Migration of Contaminants - There is some potential for contaminants to migrate through the Perched Aquifer into the swampy lowland to the north of the site or beneath adjacent properties, and to affect the quality of the water in existing or future wells. As far as can be determined there has been no particular attempt to look for contaminants in these areas. However, it is felt that this does not represent a major problem for the following reason:

1.   The individual "aquifer" units or deposits, comprising the Perched Aquifer are not believed to be continuous for more than a few hundred metres and thus lateral migration of contaminants through the Upper Unit would be expected to be restricted to the immediate vicinity of the site.

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2. In this area, the Perched Aquifer could not be considered an important ground-water resource.

3. Contaminants, migrating into the swamp from the north side of the site or from the east side from Lagoon No.1 could be intercepted by perimeter tiles, and springs could be covered, and controlled.

4. The Applicant proposes (28,v.15,p.751) to prevent the exit of contaminants by covering any "aquifer" units encountered in the excavation of refuse cells around the margin of the site with materials having relatively low values of hydraulic conductivity.

5. A relatively simple monitoring program (28,v.4,p.368), which considers the gradient and if necessary the quality of the ground water in the Perched Aquifer is proposed by the Applicant.

5.4 Contaminants in the Main Aquifer

5.4.1   Historical Quality of the Ground Water - Figure 10 is a map of the site showing the quality of the ground water in the Main Aquifer. The following information is provided on this Figure:

1.   the range in values of chloride, hardness, sulphate and specific conductance in the water samples taken from observation wells on the site and from nearby domestic wells. Where extreme values were considered to be anomalous, they were not used.

2.   a qualitative estimate of contaminant levels and sources in various observation wells and nearby domestic wells. Wells are classified as those that are believed to be contaminated by the landfill, that have no evidence of contamination or in which contamination and/or the source of contamination is very uncertain. In making this assessment consideration is given not only to the concentrations of the various parameters but also to the trends exhibited;

i.e. if the level of a particular parameter has been stable, increasing, decreasing or is erratic.

3. Wells judged to clearly have a general increasing trend in contaminant concentration are identified. These are wells OW1-76, OW2-76 and the Hutchinson and Bolton wells.

No wells showed a clear decreasing trend. For example, in OW1-73 the chloride values are decreasing but the sulphate values are increasing and in OW2-75 hardness, chloride and sulphate values are decreasing but the specific conductance and alkalinity are at a maximum.

There was a considerable amount of discussion of the 1981 Environmental Assessment Board Hearing (28) of the trends shown in the graphs of various contaminant concentrations vs time. In reviewing these graphs and these discussions it is concluded that:

a) a series of increasing or decreasing contaminant levels of less than one year in duration has little meaning.

b) in many cases an increasing trend in one parameter is accompanied by a decreasing trend in another and no unquestionably clear pattern is evident.

Thus, we have been very conservative designating that a trend is present.

The following points should be noted about the data presented on Figure 10.

a) Although there is some tendency for all of the contaminant parameters in a particular observation well to rise and fall in concert, there are a large number of exceptions to this tendency when one or more parameters do exhibit opposing trends.

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b) The "rating" or degree of contamination on Figure 10 is not related to the Ministry's drinking water criteria. In only a few cases did the indicator parameters in these wells approach or exceed such levels. Contaminants such as iron and manganese do exceed these criteria but since these parameters are not reliable indicators of contamination they cannot be used in assessments such as this. Parameters such as iron and manganese are not reliable indicators of contamination because these concentrations may be affected by factors indigenous to the wells or may be elevated above drinking water standards by natural processes.

c) The concentration of sulphate and/or hardness in samples from some of the observation wells are anomalously low. It is not certain why this occurs. It has been suggested that perhaps the low sulphate values might be related to the activity of sulphate-reducing bacteria. However, this explanation was questioned by F. Rovers (28,v.19,p.3303). It is also conceivable that these low values relate to dilution by surface water or water from the Perched Aquifer moving down the annulus of the well into the Main Aquifer.

Figure 10 indicates that contaminant levels in the Main Aquifer are elevated in the central and southwestern parts of the site and there appears to be an increasing trend in contaminant levels in the southwestern part of the site. There is also some contamination shown in the north-central part of the site at OW1-73. This is probably related to the infiltration of contaminated surface water from a pond near that well. Background levels of the various parameters were found in the northern, and eastern sections of the site, and in most of the domestic wells.

5.4.2   Water Quality in Domestic Wells Completed in the Main Aquifer

Our assessment of the water quality data for the domestic wells beside Highway 48 is as follows:

A) Believed to be contaminated from the landfill

    i) K.T. Hutchinson - definite increase in most parameters. This contamination could be associated with one or more of the following sources.

        a) contaminant migration from the site through the Main Aquifer,

        b) infiltration to the Main Aquifer of contaminated liquid wastes which moved overland from the southwest part of the site into depressions bordering Highway 48 (Section 2).

        c) infiltration to the Main Aquifer of contaminated leachate from solid wastes buried in the property south of the southwestern corner of the site (Section 2).

The possibility of contamination originating from highway de-icing chemicals (NaCl) or septic systems was raised in Section 5.2. It is not believed likely that the contamination in the K.T. Hutchinson well originated from either of these sources for the following reasons:

        a) The K.T. Hutchinson well is up the hydraulic gradient from the highway.

        b) Phenols have been present in the well on occasion. Phenols are not generally associated with de-icing chemicals.

        c) Nitrogen concentrations are not elevated, as would be expected if the contamination originated from septic systems.

ii) J. Bolton - definite increase in most parameters.
November, 1981 values show sharp decline. This
contamination is attributed to the same possible
sources listed for in the K.T. Hutchinson well.

It is not believed that this contamination
originated from highway de-icing chemicals or
from septic systems. The reasons (excepting
previous point a)) for this are listed in the
comments concerning the K.T. Hutchinson well. As
discussed in Section 5.4.4.8, the K.T. Hutchinson
and J. Bolton wells appear to be responding to
the same contaminant pulse.

B) Wells in which contamination and/or source of contamination is
very uncertain.

i) W. Baker - a gradual increase in the levels of specific
conductance, alkalinity and sulphate. There is
no corresponding increase in the levels of
chloride and hardness. The source of the
contaminants is uncertain.

ii) N. McLean - Hardness and alkalinity levels in this well
peaked in 1975 at concentrations of 329 ppm (or
perhaps 394 ppm) and 266 ppm respectively.
Subsequent analyses reflect background quality,
(hardness 235 ppm, alkalinity 225 ppm) for these
parameters. The reason for these 1975 levels is
not known.

iii) W. Mehaffey - There is definite evidence of contamination
in this well by chloride. There are also
indications of elevated concentrations of phenol
and erratic values for alkalinity. According to
D. Smith, (personal communication) of the
Ministry's Central Region office, the screen in

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this well is replaced at approximately two year
intervals. The reason for these characteristics
is not certain.

- This well was described as a "shallow well" and
affected by road salt by F. Rovers
(28,v.21,p.3550; 16,p.33).

iv) M.E. MacLean - Chloride and sulphate levels in the water
from this well have declined slightly to
background levels from levels in 1977 and 1978.
This suggests some minor contamination was
present in those years. The source of this
contamination is not known.

v) W.E. Fockler - The W.E. Focker well is assumed to be
completed in the Perched Aquifer and has been
discussed in Section 5.3.2 of this report.

C) No evidence of contamination (background water quality).
T. Devlin
R. Hoover
M. Baker (Villers, Joe's Farm)
J. Oldham
Deacon
York Sanitation Well

5.4.3   Vertical Distribution of Contaminants in the Main Aquifer

5.4.3.1 Introduction - Contaminant plumes are three dimensional.
Thus, in order to assess monitoring data and to establish maximum
contaminant levels, both the vertical distribution, in cross-
section, and the lateral or plan distribution of the contaminant
plume must be understood. Therefore the water-quality data were
examined to determine any patterns present in the vertical
distribution of contaminants, to identify the base of the
contaminant plume and to examine the ground-water flow system at the

site to see if this would be likely to cause the contaminant plume to move to depth. For example, contaminants concentrated at the top of the aquifer might suggest an origin at the surface above that location whereas increasing contaminant concentration with depth might suggest lateral movement through the aquifer from some more distant origin. This Section of the report deals with these aspects of this investigation.

5.4.3.2 Evidence of Plume Stratification - Two sets of nested wells are installed in the Main Aquifer in the contaminant plume (OW1-75, 2-75 and OW1-76, 2-76 (Appendix E). In these wells, a higher level of contamination occurs in the upper part of the Main Aquifer. An attempt was made to determine if this condition persists throughout the site, however, the data from other observation wells were not conclusive. It is not felt that data from only these two sets of nested wells can be extrapolated to describe the vertical distribution of contaminants over the entire site.

5.4.3.3 Base of Contaminant Plume - There is no conclusive evidence that water samples were obtained from beneath the contaminant plume in the Main Aquifer, either on the site or in nearby domestic wells. The highest levels of contamination in the Main Aquifer are found in Well OW16-70. The deep wells generally downgradient from Observation Well OW16-70 which obtained water samples of the contaminant plume in the southwest part of the site (Figure 10) are:

Well OW1-81 which sampled to a depth of 16 m (54 ft) and an
elevation of 294 m (965 ft) in the Main Aquifer,
Well OW1-76 which sampled to a depth of 17 m (56 ft) and an
elevation of 294 m (965 ft) in the Main Aquifer,
Well OW1-75 which sampled to a depth of 13 m (43 ft) and an
elevation of 299 m (982 ft) in the Main Aquifer,
Well OW7-73 which sampled to a depth of 6 m (19 ft) and an
elevation of 308 m (1009 ft) in the Main Aquifer,
and
Well OW3-74 which sampled to a depth of 12 m (41 ft) and an
elevation of 299 m (982 ft) in the Main Aquifer.

Samples from all these wells except OW3-74 contained contaminant levels above background. The sample from OW3-74 did not contain elevated contaminant levels but, as only one sample was obtained from this well, in 1974, the well cannot be used with any confidence to identify the base of the contaminant plume.

None of the domestic wells immediately to the south and west of the site penetrated more than 7.6 m (25 ft) into the Main Aquifer and thus no conclusions on the configuration of the plume can be drawn from the data from these wells.

The Applicant installed OW1-81, (28,v.3,p.236), for the purpose of sampling the full thickness of the Main Aquifer and in his opinion, Wells OW1-75, OW1-76 and OW1-81 sample the bottom of the contaminant plume (28,v.22,p.3629 and v.5,p.786) in the southwestern part of the site. As discussed in Section 4.3.4.4 of this review, the hydrogeology in this part of the site is subject to other interpretations. Therefore, there is no conclusive indication that the base of this portion of the contaminant plume has been determined.

5.4.3.4 Downward Movement of the Contaminant Plume - The hydrogeology of the site was examined to determine if the contaminants might be expected to move to depth in the Main Aquifer. The amount of downward movement of a contaminant plume depends in part upon the magnitude of the downward hydraulic gradient. As discussed in the Section 4.3.4.3, although this gradient has not been unequivocally defined at the site, a downward component is very likely to be present, at least in those areas where the "clayey layer" is absent, and in these areas it is expected that there is a downward component to the direction of travel of the core of the contaminant plume.

The hydrogeologic and water quality data are not adequate to assess how much downward movement has occurred. Therefore it is not known where in the contaminant plume a particular monitoring well samples. A well may sample the core, where contaminant

concentrations would be highest, or some point nearer the margin of
the plume where concentrations would be lower. The degree to which
this uncertainty has affected the assessment of the site is not
known. However, it would be particularly important to an
understanding of the monitoring data in the southwestern part of the
site to address the possibility that some of the contaminant plume
such as is for example found upgradient in OW16-70 has moved to
depth beneath the clayey unit and is therefore not sampled by the
monitoring wells in that area.

The density of the leachate or the liquid industrial waste could
also have an impact on the configuration of the contaminant plume.
This has not been addressed because the data are inadequate.

5.4.4   Assessment of a Contaminant Pulse In the Main Aquifer

5.4.4.1   Introduction - Contaminant levels in a monitoring well in
an aquifer will rise and fall in response to a pulse of
contamination moving through that particular part of the aquifer.
The water quality data from the southwestern part of this site can
be interpreted as representing such a pulse. The discussion of this
contaminant pulse represents the part of this assessment that best
fits the objectives and procedures outlined in Section 5.1. It is
presented because it illustrates the limits of our understanding of
the contaminant hydrogeology of the site and identifies a number of
questions regarding the distribution of contaminants which should be
addressed for a complete assessment.

The arrival times of this pulse, moving through observation wells in
the southwestern part of the site, are provided in Table 2. Graphs
of the chloride concentrations and the specific conductance values
versus time for this pulse are shown on Figures 11 and 12. Figure
13 is a map of the site showing the wells and waste disposal areas
pertinent to this discussion and Figure 14 is a cross-section
through the area discussed.

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TABLE 2. ARRIVAL OF MID 1970'S TO 1980 CONTAMINANT FRONTS

Well	ARRIVAL TIME						COMMENTS
	Chloride	Hardness	Sulphate	Specific Conductivity	Alkalinity	Other·	
OW16-70	late '75 or early '76	late '75	mid '76	late '76	no front		Front fairly distinct
OW7-73	late '75 or early '76	late '76	mid '76?	mid '76?	mid '76?		Front fairly distinct when data available.
OW1-75	late '76 or early '77	No front	early '78?	No front	no front		Front not well defined
OW2-75	mid '79	late '79 or mid '80	decline early '80	mid '79	mid '80		Front distinct
OW1-76	early '78	early '78 and late '80	late '80	late '77 to early '78 and late '80	late '77 to early '78 and late '80		Front configuration varies with parameter.
OW2-76	mid '80	mid '80	1978?	early '80	early '80?	phenol 1980?	Front distinct
Hutchinson	early '78	early '78?	late '77 or early '78	1977?	mid '76? and late '79		Front configuration varies with parameter.
Bolton	early '77	early '77	late '77	1977?	no front		Front distinct
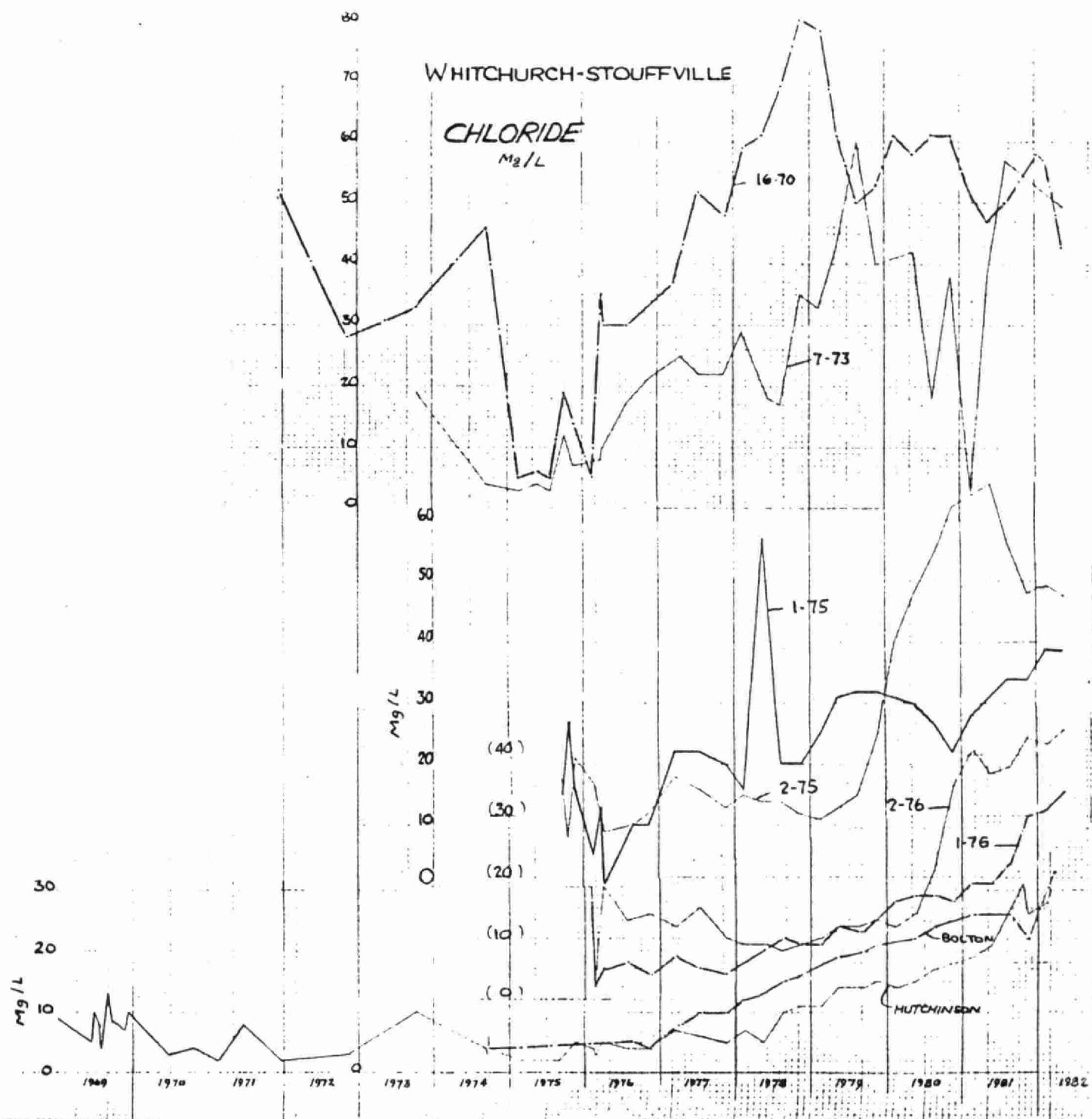
Figure 11   Contaminant Pulses Chloride
             including wells 16-70, 7-73, 1-75,
             2-75, 2-76,1-76, Bolton and Hutchinson
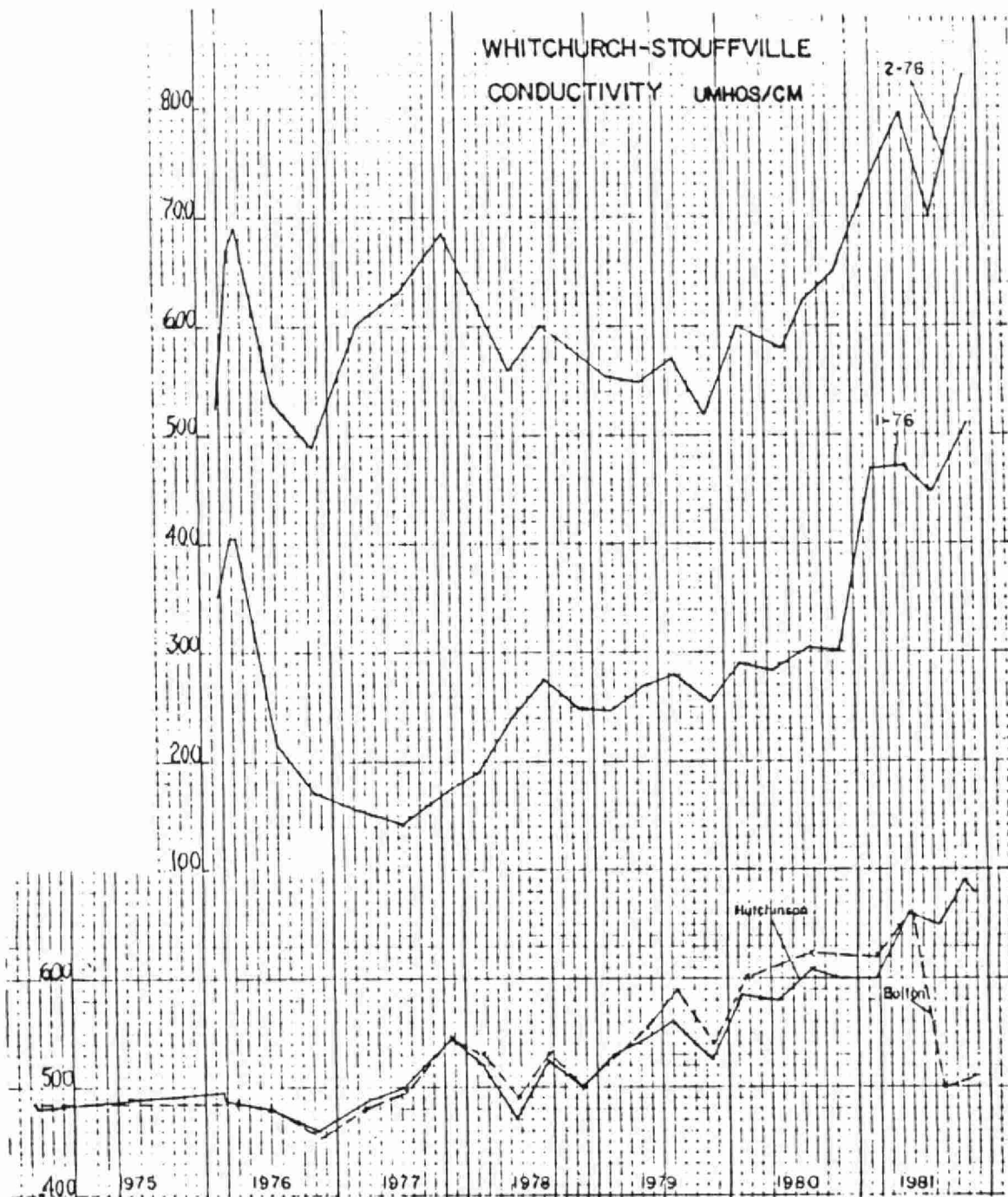
Figure 12 - Contaminant Pulses
Specific Conductance
incl. wells 2-76, 1-76, Hutchison & Bolton

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WHITCHURCH-STOUFFVILLE
CONDUCTIVITY UMHOS/CM
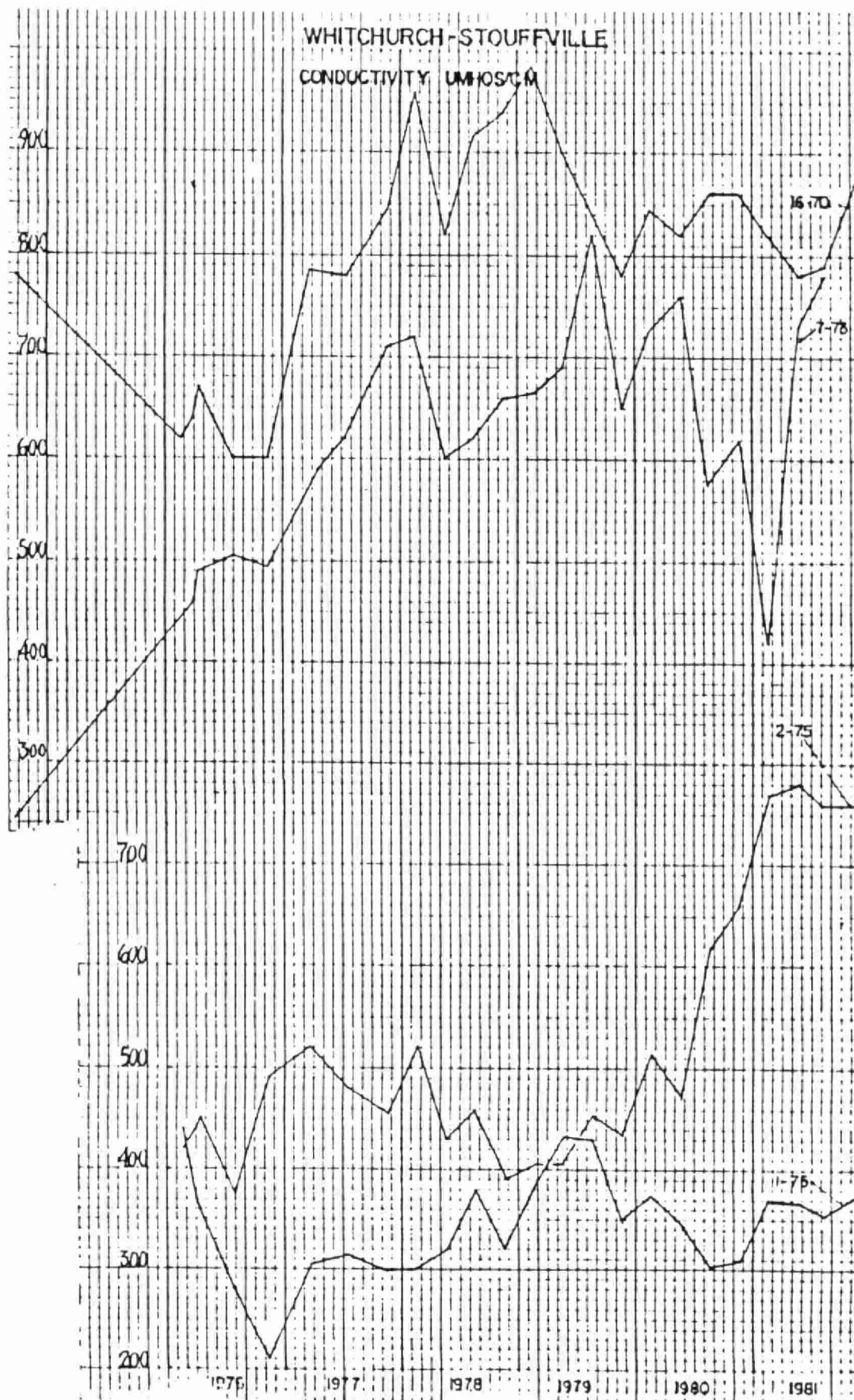
Figure 12 - cont'd - Contaminant Pulses
Specific Conductance
incl. wells 16-70, 7-73, 2-75, & 1-75.

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HWY 48

BERM

Lagoon 2

BERM

Lagoon 3

Lagoon 5

OW5-70

Lagoon 1

BERM

Lagoon 6

B

OW16-70

Lagoon 4

OW7-73

OW1-75

OW2-75

OW1-76

OW2-76

K.HUTCHISON

N

J.BOLTON

B

Figure 13. Location of Selected Observation Wells, Liquid Industrial Waste Lagoons and Cross-Section B-B'

Note: B-B' is constructed along the plan view of a flow line in the Main Aquifer. The flow line was based on the equipotential line distribution shown on Figure 5. Wells were projected onto this cross-section along equipotential lines.

0   500'   1000'

Ltd. Jan. 1970 (A)

The following aspects of this contaminant pulse were examined:

a) the source of the contamination;
b) consideration of a single contaminant pulse;
c) similarity in the slopes of graphs;
d) attenuation of contaminants;
e) the horizontal velocity of the contaminants and the hydraulic conductivity of the aquifer; and,
f) anomalies in the data

5.4.4.2 Source of the Contaminant Pulse - There are a number of possible origins for the contaminants producing this pulse. One of these, suggested by the Applicant (6,p.25; 28,v.22,p.3646), is that the pulse, which reached OW16-70 and OW7-73 in 1975 and 1976, was initiated by the filling and closure of Liquid Industrial Waste Lagoon No.1 in 1973 and 1974 (Section 2).

In the following paragraphs this suggestion is examined in detail. One purpose of this exercise is to illustrate the level of understanding of this site and the amount of speculation involved in its assessment. Although the Applicant's suggestion appears to be the most reasonable, it has deficiencies and other explanations of the origin of this contaminant pulse are possible.
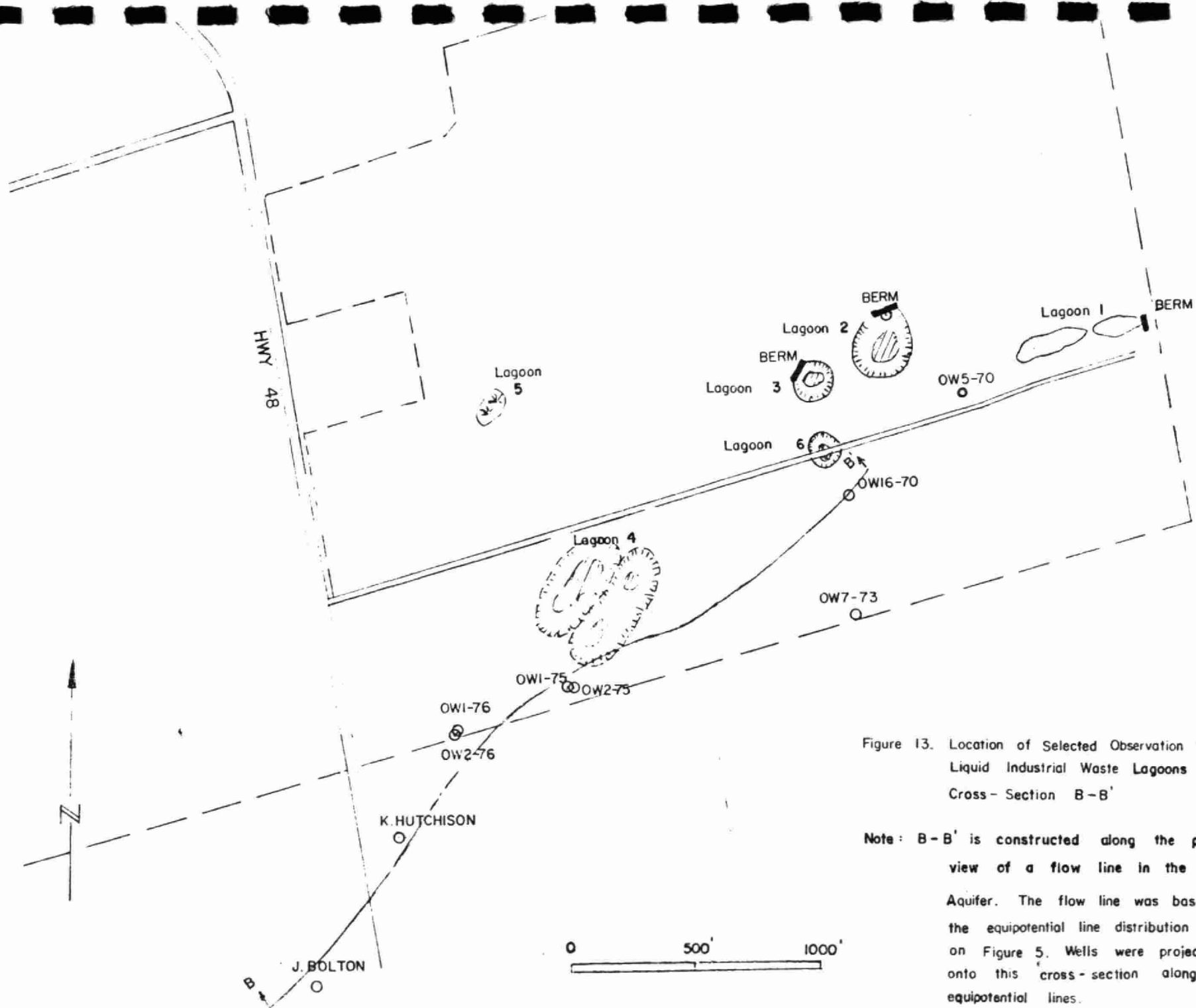
Lagoon No.1 (Fig.13) is approximately 240 m (800 ft.) to the northeast of OW16-70 and is almost directly up the hydraulic gradient (Fig.7) from OW16-70 and OW7-73.

Assuming an approximate velocity for contaminant movement of 125 m/yr to 150 m/yr as will be derived in Section 5.4.4.6 and as presented in Table 3 of this report, a contaminant front reaching OW16-70 in 1975-76 should have entered the Main Aquifer beneath Lagoon No.1 about two years earlier, or in 1973-74. If the estimate of velocity in the Main Aquifer is correct and if this contaminant pulse is related to the filling of this lagoon in 1973-74 then contaminants must have reached the Main Aquifer less than one year after this lagoon was filled with solid wastes.
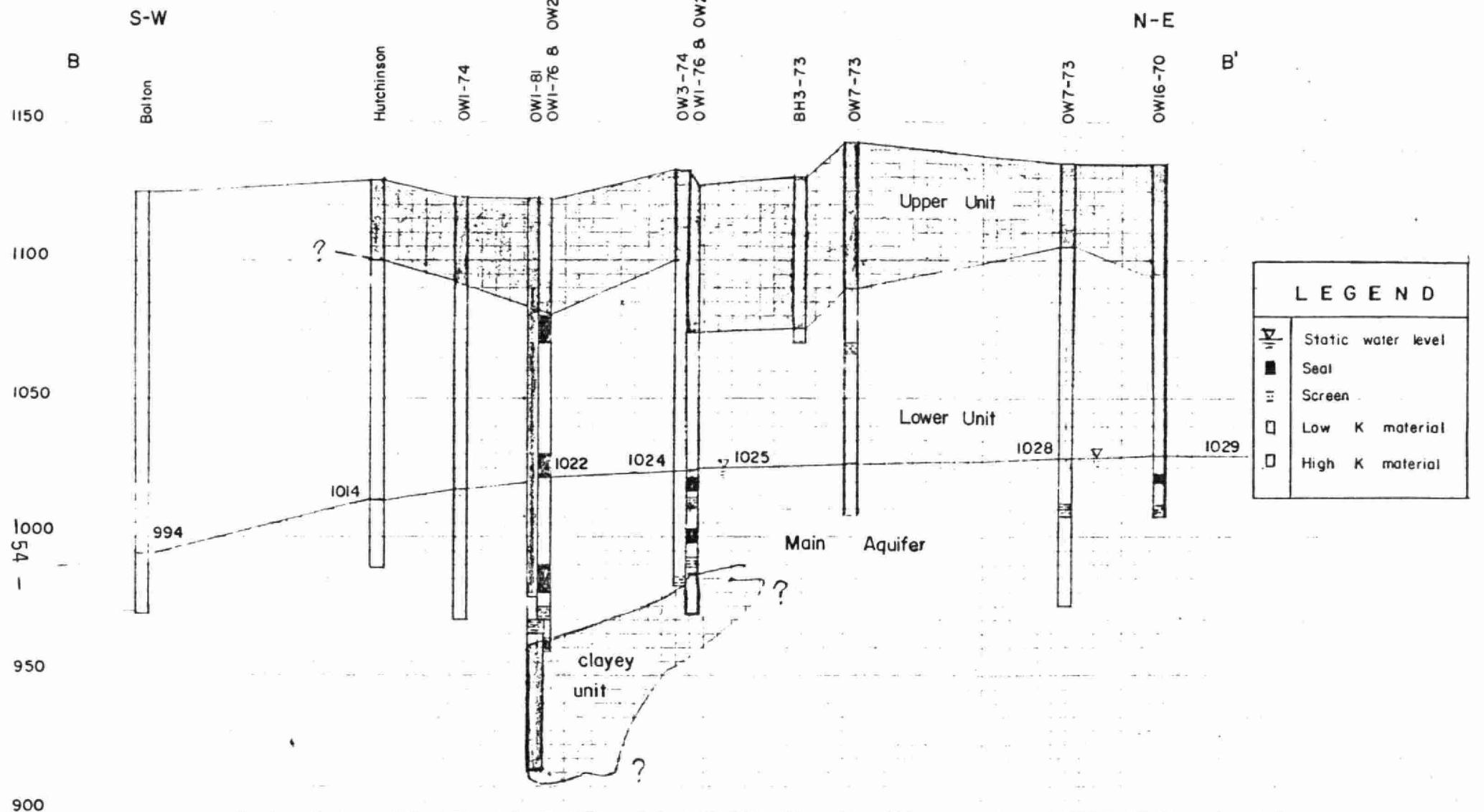
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Figure 14. Cross-section B-B' — Southwestern part of site

(Alignment shown on Figure 13)

Note: B-B' is constructed along the plan view of a flow
line in the Main Aquifer. The flow line was based
on the equipotential line distribution shown on
Figure 5. Wells were projected onto this cross-section
along equipotential lines

It is probably not reasonable to propose that there has been direct movement of fluid from Lagoon No.1 to the Main Aquifer in one year. The reason for this is as follows:

OW5-70 is the nearest downgradient obserVeration well in the Main Aquifer to Lagoon No.1, and at a distance of approximately 200 m (656 ft) to the southwest of Lagoon No.1 (Figure 11). Well OW5-70 is approximately 42.7 m (140 ft) deep and according to the well log there is about 15 m (50 ft.) of "till, silty, sandy clay" present in the Upper Unit and about 18 m (60 ft.) of unsaturated sands and gravels in the Lower Unit above the Main Aquifer. Therefore, the downward flow velocity required for contaminants to reach the Main Aquifer under these circumstances would be at least 33 m/y (110 ft./yr). As discussed in Section 5.3.3 of this report this flow velocity is far in excess of what has been proposed by other workers.

Two mechanisms seem possible to account for "rapid" movement of contamination into the Main Aquifer.

1)   If the soil column beneath Lagoon No.1 had been brought to field capacity with earlier loadings of liquid industrial wastes and leachate, any additional input of fluid at the surface would result in a corresponding discharge of contaminated fluid into the Main Aquifer.

2)   The disposal of solid waste into Lagoon No.1 opened or created a very conductive "window" to the Main Aquifer beneath the lagoon.

The first of these two mechanisms is considered to be the most reasonable.

The major difficulties in accepting Lagoon No.1 as the source of this contaminant pulse are:

1) OW5-70 lies between Lagoon No.1 and OW16-70. However, there does not appear to be any correlation between the contaminant

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pulses measured in OW5-70 and those in OW16-70. The measurement
intervals of water quality in OW5-70 appear to have been
somewhat erratic and thus there may be some question as to the
reliability of the water quality data from this well.

2) The water quality data (4,p.7) from wells put down adjacent to
Lagoon 1 to investigate the extent of seepage indicate chloride
values of 20 ppm or lower. If these low values reflect the
chloride concentration in Lagoon No.1, that lagoon could not be
the source of the 80 ppm chloride concentration in OW16-70.

There are other possible explanations for the source of this
contaminant pulse in OW16-70.

1) Lagoon No.1 had received some liquid industrial wastes by
mid-1966 (1,p.5). It is possible that the 1975-1976 contaminant
pulse in OW16-70 could be related to this earlier disposal
operation.

2) Lagoon No.6 could be responsible for this 1975-76 pulse. Lagoon
No.6 is only 40 m (130 ft) to the north of OW16-70 (Figure 13)
and using a travel velocity of 150 m/y through the Main Aquifer
allows contaminants approximately two years to move downward
through the unsaturated overburden deposits from Lagoon No.6
into the Main Aquifer after the lagoon was closed in 1973 and
1974. If sufficient lateral movement of contaminants in the
Perched Aquifer and dispersion of the contaminant plume in the
Main Aquifer is accepted to bring part of the plume from Lagoon
No.6 into OW16-70 or if the direction of the horizontal
component of the ground-water flow vector is more to the south
than shown in Figure 7, then Lagoon No.6 would appear to be a
possible source of the contaminants in OW16-70.

According to the Applicant Lagoon No.6 was used from 1967 to
1971 (28,v.5,p.764) and it is possible that this earlier
disposal in Lagoon No.6 may also be responsible for the 1975-76
pulse.

3) With only minor changes in 2) above, a similar argument can be presented to designate Lagoons No.2 and No.3 as sources of the contamination in OW16-70.

The major difficulties associated with identifying Lagoons No. 2, 3 or 6 as a source of contamination in OW16-70 are that:

a) the chloride fronts arrive at Well OW16-70 and OW7-73 almost simultaneously, (Table 2 and Figure 11) even though OW7-73 is approximately 150 m (500 ft) further south than OW16-70 from Lagoon No.6.

b) based on the water-table map of the Main Aquifer (Figure 7) OW16-70 and OW7-73 are not directly downgradient from these lagoons.

In conclusion, considering all of the data it is felt that, of the potential sources for the contamination in OW16-70 and OW7-73, Lagoon No.1 is the most probable and that, as suggested by the Applicant, the 1975-76 pulse in the Main Aquifer was initiated by the 1973-74 filling operation.

If this hypothesis is correct, it leaves in question the present disposition of the contaminants from the other lagoons and landfill areas which were not a source of the contamination in OW16-70 and OW7-73.

5.4.4.3 Consideration of a Single Contaminant Pulse - Table 2 is based on the assumption that a single input of contaminants caused the increases in contaminant levels in OW16-70 and OW7-73 during late 1975 to late 1976 and this increase is reflected in the downgradient wells in the central and southwestern parts of the site. This table presents the estimated times of arrival of various contaminant parameters in these observation wells and in the K.T. Hutchinson and J. Bolton wells. Table 2 is based on graphs such as those shown on Figures 11 and 12.

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The judgement of the arrival time of the contaminant pulse, from graphs such as Figures 11 and 12, is subjective and it is possible to modify the data on Table 2 to a certain extent and thus present a different view of contaminant flow.

The data on Table 2 indicate that, in general, contaminant fronts appear first in OW16-70 and OW7-73 in 1975 and 1976, later in 1977 to 1979 in OW1-75 and still later in OW1-76 in 1978 to 1980. This suggests a pattern of migration to the southwest across the site down the hydraulic gradient.

However, there are a number of anomalies or inconsistancies in this concept of a single contaminant pulse, migrating across the southwestern part of the site. Some of these anomalies are:

1. The J. Bolton well is much further down the hydraulic gradient from OW16-70 than the K.T. Hutchinson well (Figures 7 and 13). However, in spite of this, contaminants seem to reach these two wells at about the same time and in about the same concentrations. If these contaminants moved all the way from the site through the Main Aquifer, some additional delay and attenuation of the contaminants in the Bolton well would be expected.

2. Contaminant fronts appear to be present in the J. Bolton well in 1977 and the K.T. Hutchinson well in 1978 before they are present in upgradient onsite wells, OW2-76, and OW2-75. This is not consistent with the pattern established in the observation wells on the site.

    It should be noted that there appears to be a recent increase in the upward slope of the chloride graph of the K.T. Hutchinson well (and perhaps the J. Bolton well). If this trend develops into a distinct contaminant pulse it would fit the pattern established by the wells on the site and perhaps explain some of the anomalies associated with the present interpretation. However, it was not felt to be advisable to continually revise

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this assessment based on "trends" that have not clearly developed.

3. The contaminant front for chloride in OW2-75 and OW2-76 appears to steepen (Figure 11) with increasing travel distance through the aquifer. One would expect that dispersion would reduce the steepness of the contaminant front as it moves through the aquifer along a ground-water flow line.

4. The deeper wells (OW1-75 and OW1-76) show lower levels of contamination than the shallow wells (OW2-75 and OW2-76) even though contamination reaches the deeper wells more rapidly. One would expect that the observation wells exhibiting the highest levels of contamination would also show the arrival of the front most quickly.

5. The conservative contaminant parameters do not all move at the same velocity.

6. The general shape of the arrival curve varies with the particular contaminant.

5.4.4.4 Similarity in the Shapes of Graphs - Similarity in the shapes of water quality graphs of different wells, such as shown on Figures 11 and 12, suggests that the quality of the water in these wells is responding to the same contaminant pulse and the contaminants have moved through materials with similar hydrogeologic characteristics. The following well pairs have similar water quality graphs:

        OW16-70 and OW7-73 - chloride graph
        OW1-76 and OW2-76 - specific conductance graph
        Hutchinson and Bolton - specific conductance graph

There is no clear explanation of why some of these observation wells have similar water-quality graphs and others do not. However, it is not felt that the absence of similarity necessarily demonstrates

that the contamination originated from different sources but that it moved through materials with different hydrogeologic characteristics.

5.4.4.5 Attenuation of Contaminants - Peak values are most clearly defined in the graphs of OW16-70 and OW2-75, (Figures 11 and 12) indicating the maximum values of chloride and specific conductance obtained in samples from these wells. These peak values are compatible with a dilution of 1:1.2 and 1:1.3 after a travel distance of 430 m (1411 ft) through the Main Aquifer. Such an estimate of dilution is based on the assumption that the two monitoring wells sample the same parts of the contaminant plume (i.e., the core, the edge). We have no reason to believe that the same parts of the contaminant plume have been sampled.

The remaining graphs are less easily correlated and do not readily fit the pattern described above. For example, the peak values for specific conductance (Figure 12) and sulphate (data not presented) only partly fit a pattern of decreasing concentration with travel distance and the values for hardness do not fit this pattern at all. Therefore, the above estimate of dilution or attenuation based on the decline in the level of the chloride peak and specific conductance in the two wells is not considered reliable.

5.4.4.6 Horizontal Velocity of Contaminants and the Hydraulic Conductivity of the Aquifer - Contaminant velocities shown on Table 3 were calculated by assuming that a contaminant front (and in one case a peak) originated in OW16-70 and moved through successive wells in the southwestern part of the site down the hydraulic gradient from OW16-70. The validity of the velocity estimate depends on the degree of homogeneity of the aquifer.

The hydraulic conductivity, K, of the Main Aquifer was calculated from the relationship

$$K = \frac{v\,n}{i}$$

TABLE 3. APPARENT CONTAMINANT VELOCITIES AND CORRESPONDING HYDRAULIC CONDUCTIVITY
VALUES FOR MAIN AQUIFER

Wells	Parameter	Contaminant Front	Peak	Distance (metres)	Approximate Time** in Years	Velocity m/y	Hydraulic Gradient	K* cm/s
OW16-70 to OW1-75	Cl	x		434	1	434	0.003	1×10^{-1}
OW16-70 to OW2-75	Cl	x		427	3.5	122	0.003	4×10^{-2}
OW16-70 to OW1-76	Cl	x		556	2	278	0.004	7×10^{-2}
OW16-70 to OW2-76	Cl	x		564	4.5	125	0.004	3×10^{-2}
OW1-75 to OW1-76	Cl	x		122	1	122	0.008	1×10^{-2}
OW2-75 to OW2-76	Cl	x		137	1	137	0.007	2×10^{-2}
OW1-76/Hutch.	Cl	x		160	0	NA	0.015	NA
Hutch./Bolt.	Cl	x		221	NA	NA	0.028	NA
OW16-70 to OW2-75	SC	x		427	3	142	0.003	4×10^{-2}
OW16-70 to OW1-76	SC	x		556	1.5	371	0.004	9×10^{-2}
OW16-70 to OW2-76	SC	x		564	3.5	161	0.004	4×10^{-2}
OW2-75 to OW2-76	SC	x		137	0.75	183	0.007	2×10^{-2}
OW1-76/Hutch.	SC	x		160	NA	NA	0.015	NA
OW16-70 to OW2-75	Cl		x	427	2.5	171	0.003	5×10^{-2}

*   - hydraulic conductivity based on effective porosity, n, of 0.3

$$K = \frac{Vn}{i} \ (3.169 \times 10^{-6})$$

**   - Time determined from Table 2 and from Figures 11 and 12
Cl   - Chloride
Distance - distance along median flowline on Figure 13
SC   - specific conductance
NA   - not applicable - calculations irrational

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where v is the average linear velocity of the contaminants, n is the effective porosity of the aquifer materials and i is the horizontal hydraulic gradient. In this case the effective porosity was assumed to be 0.30 (Section 4.3.4.2). The hydraulic gradient varies as shown on Table 3 depending on which pair of wells was being considered. The value obtained for the peak between OW16-70 and OW2-75 is felt to be most representative of the average ground-water velocity.

It is realized that the hydraulic conductivity would be overestimated by considering the extreme front of the pulse. Because of dispersion the front should be travelling faster than the average ground-water velocity, which is measured by the peak.

It was possible to use peaks in only one calculation. The chloride peak in OW16-70 in late 1978 was assumed to be the same as that observed in OW2-75 in mid 1981. The calculated hydraulic conductivity value (Table 3) using these peaks is 5×10^{-2} cm/s $(1.28 \times 10^3$ igpd). Measurements from more than one pair of peaks would be necessary to place any particular reliance on this value.

There may also be a relationship between contaminant peaks in OW1-73 and OW9-73. If so this relationship is probably associated with a contaminant pulse originating from contaminated surface run-off (Section 4.3.4.1) in to a pond near OW1-73 and thus was not used in the assessment of this pulse.

It is apparent from Table 3 that there is a high degree of consistency in the values for hydraulic conductivity calculated from contaminant velocities across the site, using both chloride and specific conductance.

The interpretation of the water quality graphs (Figures 11 and 12) can be "adjusted" to give more consistency to the calculated velocities, and K values. However, in the interest of objectivity, this was not done. Contaminant fronts were identified as shown on Table 2 and then Table 3 was constructed without further modifications of Table 2.

5.4.4.7 Possible Explanations for Anomalies in the Interpretation - The anomalies in the foregoing discussions of this contaminant pulse could be explained by one or more mechanisms including;

a) variations in the direction of contaminant flow and in the velocity of the ground water and contaminants moving along various flowpaths. This would be expected in geologic deposits of this nature.

b) variation in the retardation factors for various contaminants. A retardation factor is the rate at which a contaminant migrates compared to that of the ground water.

c) the disposal of contaminants at different parts of the site at different times and the arrival of these contaminants in different parts of the Main Aquifer at different times. In addition there has been some surface runoff of contaminants from the site. (Section 2 this report.) and

d) incomplete understanding of which part of the contaminant plume is sampled by a particular monitoring well. This could lead to, for example, the dilution of contaminated water entering the well bore from one section of the well by uncontaminated water from another section. It is very unlikely that any of the monitoring wells sample only the most concentrated part of the contaminant plume.

All of the anomalies identified in this part of the assessment document could probably be explained by one or more of the foregoing mechanisms. However, in the absence of supporting data, such explanations are speculative and cannot be relied upon in understanding the contaminant hydrogeology of the site.

5.4.4.8 Conclusions and Questions Raised by the Assessment of the Contaminant Pulse - The consistency in the values for hydraulic conductivity of the Main Aquifer calculated from velocities of the contaminant fronts as they move across the site is relatively strong

evidence that the monitoring wells sample the same contaminant front related to a single episode of contaminant discharge. The most likely source for the contaminants would appear to be Lagoon No.1 with the pulse originating during the filling and closure of this lagoon in 1973 and 1974. This consistency does not extend offsite into the K.T. Hutchinson and J. Bolton wells and these wells may not be a part of this "system".

There are similarities in the graphs of the contaminant pulses in the (OW16-70, OW7-73), (OW1-76, OW2-76) and (K.T. Hutchinson, J. Bolton) pairs of wells. This suggests that, within these pairs of wells, the contamination is from the same source.

The foregoing raises major questions:

a)   Where is the contamination that did not originate in Lagoon No.1? How meaningful is the present monitoring?

b)   What is the source of the contamination in the K.T. Hutchinson and J. Bolton wells? Can the "Group 1" monitoring wells (OW1-75, OW2-75, OW1-76, OW2-76) on the site, be relied on to provide advance warning (28,v.3,p.247, and 12,p.11) of contamination in the domestic wells.

5.4.5   Additional Comments

1)   Water quality data are available from a series of samples collected in the late 1960's and early 1970's from observation wells in the Main Aquifer in the south central part of the site and from private wells. These data indicate levels of some contaminants slightly higher than later levels, which are assumed to represent background. Wells showing the elevated levels of contaminants are:

        W. Baker   - chloride (17 ppm in 1968)
        K.T. Hutchinson   - chloride (17 ppm in 1968)
                              - phenol (3 ppb in 1970)

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M. Baker
(Viller's, Joe's Farm) - chloride (28 ppm in 1968)

York Sanitation   - phenol (6 ppb in 1970)

OW5-70   - chloride (80 ppm in 1970)
                       - phenol (25 ppb in 1970)
                         (other parameters also elevated)

OW16-70   - chloride (60 ppm in 1970)
                       - phenol (250 ppb in 1970)

The significance of this early contamination is not known. The
Applicant believes that most of the liquid industrial wastes
disposed of at the site prior to 1970 have moved off the site
and dispersed into the environment (28,v.6,p.894.). Therefore
this early contamination may have originated from the liquid and
solid waste disposal operations that were terminated in 1970.
It might also be attributed, at least in part, to factors such
as sampling and analytical techniques or well construction
methods.

Early reports (4,p.8, and 5, p.14) on the site did not attribute
these elevated values in the domestic wells to contamination;
probably because they do not "stand out" in the absence of the
subsequent decline to the lower background levels shown by later
analyses.

2) The slope of the water-quality graph at a contaminant front is
   steep in some wells and relatively gentle in others. One might
   speculate that the contamination originating from the disposal
   of wastes in or over relatively conductive parts of the Upper
   Unit, would be reflected by water-quality graphs showing steep
   increases in contaminant levels (e.g. chloride in OW16-70 and
   OW2-76). Contamination originating from those wastes disposed
   in or over the less conductive parts of the Upper Unit would be
   reflected by graphs showing more gradual increases in

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contaminant levels (e.g. chloride in OW1-76 and K.T. Hutchinson well). This possibility was examined but could not be resolved.

3) The possible impact of the waste disposal operation on the water quality at the Stouffville Municipal Wells to the south has been estimated by three workers. These estimates listed below are based on the worst case assumption that all of the contaminants from the landfill reach the municipal wells.

    1.   International Water Supply Ltd. (8, p.6) estimated that "should all the leachate migrate towards these (municipal) wells .... the resulting water (leachate) ..... would only be about 2% of the discharge ....."

    2.   Hydrology Consultants Limited. (5, p.12) estimated that "By dilution alone the concentration (of contaminants) should be reduced by a factor of at least 50."

    3.   R.W. Gillham (30,p.8) estimated that "... 3.6% of the water from the municipal wells would have originated in the landfill".

It will probably be much easier to determine if dilution is greater than the necessary amount than to determine the amount of dilution. This is addressed in Section 5.3.1.

The data that are available are not adequate for us to modify these estimates.

It is our opinion that the amount of dilution that contaminants from this site would receive cannot be practically addressed further until the nature and concentration of the contaminants in the waste and the contaminant plume have been determined, and, based on this, the amount of dilution (or attenuation) that is necessary has been decided.

It should be noted that analyses of the quality of the water in the municipal wells do not indicate that there has been any effect on these wells by contamination from the landfill.

5.4.6   Data - Assessment and Needs

5.4.6.1 Limitations of Existing Data - Despite the fact that there is an exceptionally large amount of hydrogeologic and chemical data available for this site, the movement of contaminants is not clearly understood. The data indicate that there is a general relationship between those monitoring points containing contaminants, the general ground-water flow direction and the areas of the site which received waste. However, it has not been possible to develop a coherent overall pattern showing specific contaminant flowpaths and the attenuation of these contaminants as they migrate with the ground water.

5.4.6.2 Further Assessment of Existing Data - Our analysis of contaminant migration dealt primarily with water-quality data on chloride, hardness, sulphate and specific conductance. Other parameters, such as phenol, were examined to a lesser extent. It is possible that further work with the existing data, perhaps dealing with these other contaminants, or with the ratio of one contaminant to another might shed some further light on contaminant migration at this site. However, it is our opinion that further manipulation and study of these data will not provide information which would allow an appreciably better judgment of the environmental impact of the site.

5.4.6.3 Needs for Additional Data - A major objective of this assessment was to identify the goals or the performance which should be achieved in the hydrogeologic investigation of the site to provide a high level of assurance that the environment can be protected. It was not an objective to direct the investigation of the site by specifying the methodology to be followed in an investigation, as for example, the numbers and locations of additional borings.

To provide assurance that the environment will be protected requires
determining if natural attenuation and contingency measures will be
sufficient to reduce the concentration of the various contaminants,
that have been or will be released into the ground water, to levels
that will not impair the present or potential use of the ground
water beneath adjacent property. To achieve this goal a coherent
picture of contaminant distribution and attenuation beneath the site
must be developed as discussed in Section 5.1 of this document.
This goal was not achieved.

At this site the principal deficiencies are:

a) The contaminant plume(s) throughout its entire thickness in the
    Main Aquifer has not been described and delineated.

b) The contaminants in the deposits overlying the Main Aquifer and
    in the waste itself have not been described and delineated.

In our judgement a high level of assurance in the appraisal of such
factors as dilution and contaminant attenuation must be based on
several supporting sets of measurements showing the reduction in
contaminant levels in the existing contaminant plume, rather than
inferred indirectly from water balance or underflow calculations.

There are a number of reasons why an adequate hydrogeologic
assessment of this site may be very difficult. These include the
following:

1. There are no clear records of the types of wastes and disposal
    locations. Thus, an assessment of potential contamination will
    require an extensive network of monitoring wells to locate any
    leachate and associated contaminants and exhaustive chemical
    analyses to determine their composition.

2. The type of hydrogeologic environment present at this site is
    likely to be difficult to explore and to assess. The
    ground-water flow system appears to be complex, exploratory

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drilling and monitoring wells must penetrate to considerable depths and some boreholes will probably have to be installed through solid wastes. Thus relatively large numbers of complex installations may be necessary for a clear understanding of the conditions at the site.

For these reasons serious consideration should be given to the total extent and the costs involved in completing any further assessment to determine if this is practical.

SUMMARY AND CONCLUSIONS

Background - The York Sanitation No.4 Landfill site is located in
the Oak Ridges Kame Moraine complex. The site is in an upland area
which slopes abruptly into a lowland immediately to the north and
more gently downslope towards the south. To the north of the site,
the lowland area is swampy, to the east it contains Musselman Lake,
and to the west forms the headwaters of the East Branch of the
Holland River.

Two hydrogeologic units have been defined beneath the site, the
Perched Aquifer in the near-surface deposits and below that, the
Main Aquifer. The Perched Aquifer consists of relatively thin,
conductive sand units of limited lateral extent in a matrix of less
conductive glacial till. Ground-water flow in the Perched Aquifer
generally follows the surface topography. In the northern part of
the site, flow is toward the lowland to the north and northeast and
in the southern part of the site, flow is to the south. Although
the Perched Aquifer is used by some shallow domestic wells in the
vicinity, it is not considered to be a major water resource.

The Main Aquifer underlies the Perched Aquifer and, beneath most of
the site, it is separated from the Perched Aquifer by unsaturated
deposits of sand and silty sand. Recharge to the Main Aquifer is
from the Perched Aquifer and also most likely from the swampy
lowland to the north of the site. Ground-water flow in the Main
Aquifer beneath the site is from the north and the east toward the
south and the west. The Main Aquifer is a major ground-water
resource used by many domestic wells and by the Stouffville
municipal wells.

Waste disposal operations began at this site in 1962 and both liquid
industrial wastes and solid wastes were received until 1970. There
is little information available on the amount and composition of
wastes disposed of at the site during this period. Since 1970, the
site has received solid municipal and industrial wastes.

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There are considerable ground-water quality data available that show that contamination from waste disposal activities is present in both the Perched Aquifer and the Main Aquifer and is moving with the ground water in these aquifers.

Site Impact

1. Contaminants which are believed to be associated with the disposal operations are present in two private wells in the Main Aquifer to the south and west of the site. Except for parameters naturally elevated in this aquifer, as for example iron, the contaminants are well within the limits considered acceptable for domestic water consumption by the Ministry.

2. Ground-water flow in the Main Aquifer appears to be from Musselman Lake toward the site and therefore the potential for the contamination of domestic wells in the vicinity of Musselman Lake by contaminants from the disposal site is considered to be very small.

3. Because of the hydrogeologic characteristics of the deposits in the Perched Aquifer, it is extremely unlikely that water entering this system on the site moves more than a few hundred feet away from the site boundaries before infiltrating downward. Thus, the evidence is very strong that contaminants from the site cannot move to wells in the vicinity of Musselman Lake.

4. There is a strong possibility that any future, major development of water resources on adjacent properties to the south, to the west and perhaps to the east would be impacted by contaminants from the landfill. Cones of capture of future adjacent wells could extend beneath the site and thus some part of the well yield would consist of contaminated water originating from beneath the site.

Present State of Knowledge and Needs

This landfill is in a complex hydrogeologic environment and a detailed and comprehensive hydrogeologic assessment is thus very difficult. This is further complicated by the fact that records of waste disposal at the site are incomplete. Therefore, despite the large amount of effort expended in monitoring this site, its hydrogeology is still not well enough understood to fully assess the present and potential impact of the site on the ground waters beneath adjacent properties. To accomplish this, additional information would be required in three broad areas. These are:

1) the present composition, distribution and concentrations of contaminants in the landfill and in the soils above the Main Aquifer,

2) the present composition, distribution and concentration of the contaminant plume, including its base, in the Main Aquifer and,

3) the amount of natural attenuation provided by the site and the amount of attenuation required to achieve an acceptable level of contaminant discharge from the site.

Without a clear understanding of the foregoing, it is not possible to properly address:

a) a full and meaningful interpretation of the present monitoring data,

b) the potential magnitude of offsite contamination from the existing landfill and from any expanded landfill,

c) the applicability of any proposed contingency plan,

d) a monitoring program to fully describe the contaminant plume and,

e) the maintenance and restoration activities which could be required in conjunction with or as a result of present and any future operations.

The resources and time that would be required to gather this information are not addressed in this assessment. These could be substantial.

A P P E N D I X   A
LIST OF DOCUMENTS REVIEWED

APPENDIX A

LIST OF DOCUMENTS REVIEWED[1]

1. Ontario Water Resources Commission, report on field
   investigations "Possible Surface Water Pollution from the
   O. Pellett Sanitary Landfill Site" by C.L. Young,
   Engineering Technician, July 15 and 20, 1966.

2. Ontario Water Resources Commission, Hydrogeologic Investigation
   of Waste Disposal Site by L.G. Bryck, Surveys and Projects
   Branch, October 7, 1966.

3. Ontario Water Resources Commission, Report on field
   investigations, Examination of Disposal Wastes Nos.14657
   to 14661 by H.L. Vanesche and P.I. Diosady, Dec. 1, 1969,
   Jan. 20, 1970.

4. Preliminary Report on Investigation of Liquid Waste Disposal
   Site, Township of Whitchurch Lot 14, Concession 8, January
   15, 1970, J.P. Nunan, Hydrology Consultants Limited.

5. Summary Report on Investigation of Liquid Waste Disposal Site,
   Township of Whitchurch Lot 14, Concession 8, September
   1970, B.W. Beatty, J.P. Nunan, Hydrology Consultants
   Limited.

6. Final Development Plan and Operational Procedure for York
   Sanitation, Sanitary Landfill Site No.4. June 1974, F.A.
   Rovers and J.P. Nunan, Hydrology Consultants Limited.

7. Review of Development Plan and Operational Procedure for York
   Sanitation Landfill Site No.4 as Prepared by Hydrology
   Consultants Ltd. International Water Supply Ltd., V.R.
   Dixon, August 15, 1974.

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[1]   Not in technical format.

8. Review of Development Plan and Operational Procedure for York Sanitation Landfill Site No.4 International Water Supply Ltd. V.R. Dixon, November 4, 1974.

9. Addendum to the Final Development Plan and Operational Procedure for York Sanitation Sanitary Landfill No.4, November 1974, F.A. Rovers, Hydrology Consultants Limited.

10. Quarterly Progress Reports 1, 2 and 3 from April 1975 to November 1975 by F.A. Rovers, Hydrology Consultants Limited.

11. Quarterly Progress Report and Progress Reports from May 1976 to 24 August 1981 by Conestoga-Rovers and Associates Limited.

12. Response to Condition No.11 Provisional Certificate of Approval No.A 230701 August 1978 Conestoga-Rovers and Associates.

13. Addendum 1 - Response to Condition No.11, Provisional Certificate of Approval No.A230701, January 1979, Conestoga-Rovers and Associates.

14. Survey of Inactive Waste Sites. Central Region files Site 7001, June 1979.

15. Investigation of Pond Contamination, July 1979 Conestoga-Rovers and Associates.

16. Development Operation and Management Plan, May 1980, Volumes 1, 2 and 3, Conestoga-Rovers and Associates.

17. Aquifer Performance Test, York Sanitation Water Supply Well, January 1980, Conestoga-Rovers and Associates.

18. Environmental Assessment Board Public Hearing Application of York Sanitation Limited to extend site. Witness statement of the corporation of the Town of Whitchurch/Stouffville, exhibt 54A, February 4, 1981.

19. Response to Town of Whitchurch/Stouffville, February 1981, Conestoga-Rovers and Associates.

20. Addendum 1 response to Town of Whitchurch/Stouffville, March 17, 1981, Conestoga-Rovers and Associates.

21. Proposed Plan and Investigation Lagoon L-5 and Vicinity, September 1981, Conestoga-Rovers and Associates.

22. Alternative Water Supply Contingency Plan, October 1981, Conestoga-Rovers and Associates Limited.

23. Well Installation Details, Lagoon L5 and Vicinity, November 1981, Conestoga-Rovers and Associates Limited.

24.   DUPLICATE - REMOVED

25. Environmental Hearing Board Report on the 1975 hearing, Oct. 8, 1975.

26. Transcripts of the Environmental Hearing Board 1975 hearing.

27. Environmental Assessment Board Report on the 1981 Hearing, Nov. 19, 1981.

28. Transcripts of the Environmental Hearing Board, 1981 hearing.

29. Letter March 16, 1982, Diane Jones to Dr. George Hughes.

30. Hydrogeologic Evaluation of York Sanitation Landfill Site. No.4, by R.W. Gillham, Exhibit No.54, 1975 Environmental Hearing. Prepared for the Corporation of the Town of Whitchurch-Stouffville.

31. Pumping Test on Production Well at the Town of Stouffville December 1965 (PW @ TW #8 350 IGPM) Scope.

32. Report on a Pumping Test for the Town of Stouffville June 1965 (TW #5 466 IGPM) Scope.

33. Report on a Pumping Test for the Town of Stouffville, August 1965, (TW #10 50 IGPM)

34. Industrial Water Resources of Canada Upper St. Lawrence River-Central Great Lakes Drainage Basin in Canada, 1954, J.F.J. Thomas; Water Survey Report No.3 p.186, Dept. of Mines and Technical Surveys, Ottawa.

35. Ground Water Resources of the Duffins Creek-Rouge River Drainage Basins, 1977; U. Sibul, K.T.Wang, and D. Vallery Water Resources Report 8, Ontario Ministry of the Environment.

36. Gwyn, Q.H.J. and DiLabio, R.N.W., 1973: Quaternary Geology of the Newmarket Area, Southern Ontario; Ontario Div. Mines, Prelim. Map P.836, Geol. Ser., scale 1:50,000.

37. Brown, D.M., G.A. McKay, and L.J. Chapman, 1980: The Climate of Southern Ontario, Climatological Studies, Number 5, Environment Canada, Atmospheric Environment Service, Canadian Government Publishing Centre, Supply and Services Canada, Hull, Quebec, K1A 059.

38. Drainage Works, York Sanitation Company Limited Site No.4. Conestoga-Rovers and Associates. Undated but probably the fall of 1977.

39. Response to Condition No.6, October 1981, Connestoga-Rovers and Associates Ltd.

40. A. Mellary, Central Region, 1980, Memorandum to P. Isles, Environmental Approvals Branch, Feb. 18, 1982.

41. P.K. Lee, Gartner Lee Assoc. Ltd., 1982, Letter to the Town of Whitechurch/Stouffville, Jan. 27, 1982.

42. Bentall, R., (1963) Methods of determining permeability, transmissibility and drawdown U.S. G.S. Water Supply Paper 1536-1 pp.243-341.

43. Todd, D.K., (1980), Groundwater Hydrology, 2nd Ed., John Wiley and Sons Inc.

APPENDIX   B

DATA AND SAMPLING METHODS

APPENDIX B

DATA AND SAMPLING METHODS

Sources of Data

The bulk of the data used in this assessment were obtained from
reports submitted to the Ministry and exhibits submitted to the
Environmental Assessment Board at the 1981 Hearing. Some additional
data were provided directly by the Applicant's consultant,
Conestoga-Rovers and Associates. In particular these data included:

1) Up-dated logs of the observation wells and soils borings

2) Up-dated tabulations and graphs of selected water-quality
data as of January 1982.

Data from the Ministry's files of water-well records were also used
to address the regional hydrogeology and the municipal wells.

There is some uncertainty about the depth and the aquifer used by
three domestic wells to the west of the site and some explanation is
necessary before proceeding further. The following information was
provided by A. Mellary of the Ministry's Central Region.

There are two wells on the W.E. Fockler property. One of these is a
dug well from which water quality samples are regularly taken as a
part of the site monitoring program. No log is available for this
well nor are there any measurements of the depth or the water
level. This well was described by the owner in his submission to
the 1975 Hearing as a "deep dug well" (26,v.11,p.2209). There is
also a drilled well on the W.E. Fockler property and a log for this
well is presented in Appendix C of the Applicant's 1980 submission
(16,v.2). (The water level elevation used on Figure 7 was obtained
from this log.) There are no water quality data from this well.

There is no log for the W. Mehaffey well. This well is sampled as a part of the routine water quality monitoring program.

A. Mellary believes it more likely that the drilled well on the W.E. Fockler property and the W. Mehaffey well are completed in the Main Aquifer and the dug well on the W.E. Fockler property is completed in the Perched Aquifer[1].

Samples of Earth Materials

Sampling methods and sampled intervals were noted on the borehole logs submitted by the Applicant. These data were used in the assessment of the hydrogeology of the site. Our comments on the data are as follows:

1. There is a certain amount of professional judgement involved in the description of the earth materials penetrated by a borehole. This judgement becomes most important when the boring method and sampling procedures are not particularly well suited for obtaining representative samples. As a general rule, where it is important to obtain an accurate knowledge of the geologic materials, the more sophisticated sampling techniques such as split-spoon sampling are required. Where less certain sampling methods are employed, as for example, the collection of grab samples from rotary drilling, the assessment of the type of material present may vary according to the professional judgement of the assessor.

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1   Data provided in September, 1982 by J. D'Cruz of the Ministry's Central Region indicate that the W.E. Fockler dug well is 6.15 m (20.5 ft.) deep and had a static water level of 320.3 m (1051 ft.) on June 29, 1982 and of 319.7 m (1049 ft.) on September 8, 1982. This dug well is located in the lowland north of the house. The drilled well is buried beneath the driveway in front of the house.

It is felt that this judgement factor is at the root of the uncertainty regarding the thickness of the Main Aquifer in the southwest corner of the site and the definition of the base of this Aquifer. Apart from this, no serious differences were noted in the interpretation of the geology.

2. The Applicant has classified the geologic materials described on the logs of the soils borings according to texture, using the "Unified Soil Classification System". It was noted in the review of the Applicant's submissions that his use of this classification is somewhat interpretive. Therefore, it was necessary to use the description on the original borehole logs in our review of the cross-sections submitted by the Applicant. Alternate interpretations of the soils data should be taken into consideration in future work on the hydrogeology at this site.

Hydraulic Conductivity Measurements

Three methods were used to obtain estimates of the hydraulic conductivity of the earth materials at the site.

1) Estimates based on falling head, input or slug tests.

These estimates were made (5, Table 2) during the drilling of observation wells and according to J. Nunan, (personal communication) they were probably done by filling the boreholes to the top and measuring the rate of decline in the water level. They must be considered as semi-quantitative in that "standard" procedures for conducting these types of tests were not followed. Comments by the Applicant's consultant (26,v.4,p.506) indicate that he would agree.

A slug test was conducted on OW10-70 (11, Aug., 1978, p.6) which yielded a hydraulic conductivity of 7.1×10^{-4} cm/s $(1.8 \times 10^{1}$ igpd/ft$^{2})$. There was silting within this well, and the results may not be representative.

2) Estimates based on the grain-size distribution in a soil sample (11, 12).

3) Estimates based on laboratory tests with a falling head permeameter (11, 12).

4) Various pumping tests on the Municipal wells and the York Sanitation office well. These are discussed in Appendix D.

We can see no need for additional data on the hydraulic conductivity of the materials at this site at this time.

Samples of the Ground Water

Ground-water sampling procedures included "cleaning" the well by bailing various volumes of water ranging from less than one to several well volumes before obtaining samples for analysis. The relative consistency in the results of the analyses indicates that the sampling procedure has been adequate. Sampling procedures were developed by the Applicant in conjunction with the Central Region of the MOE. They are described in the Applicant's submissions (16,v.2,App.D).

Measurement of Ground-Water Levels

Water levels were measured before water-quality samples were taken. Comments on the response of the water levels to this sampling are tabulated in the attached memorandum from the Applicant's consultant. Information such as this is useful in assessing the significance of water-quality and water-level data. For example, there are some questions (Section 4.3.4) as to the interpretation of the water-level data from Well OW4-73. In the table accompanying the Applicants' Consultants' memorandum (following this page) it is noted that it is difficult to obtain sufficient water from Well OW4-73. This suggests that the screen or the well bore may be partially plugged or that the well samples only fine textured materials with low hydraulic conductivity. The construction of this

~ ? /\

Consulting Engineers

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March 31, 1982

Project No. 9-0184

Dr. George Hughes
Resources Assessment Unit
Hydrology and Monitoring Branch
Ministry of the Environment
135 St. Clair Avenue West
Toronto, Ontario
M4V 1P5

Dear Dr. Hughes:

Re: York Sanitation Site No. 4 - Bremner

The attached table presents the following information for
OW16-70, OW1-73, OW4-73, OW6-73, OW7-73, OW9-73, OW1-74, OW1-75,
OW2-75, OW1-76, OW2-76, OW1-78, OW2-78, OW3-78, OW6-76, OW1-80,
OW2-80 and OW3-80.

i)   Typical Volume of Water Contained in Well (Liters):

   - determined by examining historical water levels since 1977
     (where appropriate) and calculating water volumes in
     accordance with well diameter and well depth as presented
     in the revised well logs.

   - these volumes typically exhibit some seasonal fluctuation.

ii)   Typical Volume of Water Bailed Prior to Sampling (Liters):

   - the usual volumes of well water removed by hand-bailing
     prior to sampling.

   - these values do not include the water collected and
     submitted as a sample.

   - fluctuations in volumes generally reflect changes due to
     variations in water levels.

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iii) <u>Well Volumes Bailed Prior to Sampling:</u>

- this number expresses the volume of water removed by
  bailing in terms of well volumes rather than liters.

iv)   <u>Comments:</u>

- the information presented in Table 1 and field notes have
  been examined in order to comment on the well performance.

- it should be noted that the comments are not based on
  actual field recovery tests.

- it should be noted that, in the cases of OW1-75, OW2-75
  and OW3-80, less than one volume of well water is bailed
  due to historical continuity not because of poor well
  performance.

Should there be any questions regarding the above, please do not
hesitate to contact me.

Yours very truly,

CONESTOGA-ROVERS & ASSOCIATES LIMITED

Diane Jones, B.Sc.

DJ:dp
Encl.

c.c.   Mr. Ron Poland, P. Eng.

TABLE 1

Well No.	Typical Volume of Water in Well (Liters)	Typical Volume of Water Bailed Prior to Sampling (Liters)	Number of Well Volumes Bailed Prior to Sampling	Comments
OW16-70	3 - 4	8 - 15	2 - 4	- consistently bailed at least 2 volumes - appears to recharge quickly
OW1-73	5 - 10	3 - 12	0.5 - 1.0	- usually no problem getting sample
OW4-73	1 - 2	0.5 - 2	0.3 - 1	- water level has been decreasing due to lowering of perched water table - rarely get complete sample
OW6-73	2 - 2.5	2 - 5	1 - 2	- appears to recharge quickly
OW7-73	6 - 7	15	2	- appears to recharge quickly
OW9-73	2 - 3	5 - 15	1.5 - 7	- appears to recharge quickly
OW1-74	1 - 8	1 - 8	0.5 - 1	- water volume seasonal - recharge appears to be slow - rarely get complete sample
OW1-75	22 - 23	8 - 15	0.4 - 0.7	- appears to recharge quickly
OW2-75	8.5 - 10	6 - 15	0.6 - 1.7	- appears to recharge quickly
OW1-76	28 - 30	39 - 40	1.3	- appears to recharge quickly
OW2-76	7 - 8	15	2 - 2.5	- appears to recharge quickly
OW1-78	1.0 - 2.5	0.5	<0.5	- recharges very slowly

TABLE 1 (continued)

Well No.	Typical Volume of Water in Well (Liters)	Typical Volume of Water Bailed Prior to Sampling (Liters)	Number of Well Volumes Bailed Prior to Sampling	Comments
OW2-78	8 (?)	0.5	<0.1	- recharges very slowly - volume of water bailed in 1979-1980 = 4 - 8 Liters but now typically only 0.5L - often difficult to collect a full sample - well may be silted up somewhat
OW3-78	2.5 - 3	5 - 7.5	2	- recharges slowly, probably due to poor well installation
OW6-78	15 - 16	12 - 15	0.8 - 1	- recharges less quickly than would be expected
OW1-80	3 - 4.5	0.5 - 5	<0.2 - 1	- water levels decreasing as perched water level decreases - well appears to recharge slowly - presently remove only 0.5 Liters prior to sampling
OW2-80	9 - 10	15	1.5	- appears to recharge rapidly
OW3-80	21	15	0.7	- well is silted up somewhat but there is no problem bailing and collecting sample

well is shown on Figure 8. This question has not been resolved with certainty and thus the data from Well OW4-73 can be questioned.

Table 1 in this Appendix presents construction data for the observation wells on the site. This allows some independent assessment of the reliability of the data obtained from these wells. For example, wells completed without a seal in the annulus of the borehole above the screen may not measure the ground-water potential near the elevation of the well screen.

Although there are some questionable water-level data associated with the ground-water mound, for the most part water levels at the site conform to a reasonable pattern of ground-water flow.

Regional Mapping

Regional maps are of value in assessing major features associated with regional flow systems. However they are of limited value in the assessment of small local hydrogeologic features that are not obvious or pronounced but may be critical to the assessment of such things as landfills.

Regional maps such as Figure 6 are based on the assessment of drilling records of domestic and municipal wells in the Ministry's files. These wells were not drilled to provide hydrogeologic data for assessments such as this and thus the log of each well must be examined carefully by the mapper to determine if, based on professional judgement, the data it provides are acceptable.

The first step in this procedure is to reject data from wells that are unsuitable for obvious reasons (e.g., location uncertain, log incomplete, well not completed in aquifer of interest). After this has been done, additional wells, perhaps 20%, must still be "rejected" because the data from them are anomalous. Thus, maps such as this incorporate a considerable amount of judgement.

TABLE 1: APPENDIX B

OBSERVATION WELL DATA
(see legend for explanation of terms)

| Boring | Boring Method | Sampling Method | Material in Bore Hole Annulus | | |
			Screened Interval	Seal	Backfill
BH 1B-70	SSA	ND	ND	ND	ND
OW1 -70	WB	SS-7	S	Cmt	Cmt
OW2 -70	WB	SS-2	S	Cmt	Cmt
OW4 -70	WB	SS-3	S	Cmt	Cmt
OW5 -70	SSA-WB	WS-7	S	NF	S
OW6 -70	SSA	Ag S-4	S	ND	S
OW7 -70	SSA	Ag S-4	S	ND	S
OW8 -70	SSA	Ag S-5	S	ND	S
OW9 -70	SSA	ND	S	ND	S
OW10-70	SSA	ND	S	ND	S
OW11-70	SSA	ND	S	ND	S
OW12-70	SSA	ND	S	ND	S
OW13-70	SSA	ND	S	ND	S
OW14-70	SSA	ND	S	Cmt	Cmt
OW15-70	SSA	ND	S	ND	S
OW16-70	SSA & MR	SS-2	NS	Cmt?	Cmt
OW1 -73	HSA?	ND	S	ND	Cu
OW2 -73	HSA?	ND	S & Gvl	ND	Cu
OW3 -73	HSA	ND	S & Gvl	ND	Cu
OW4 -73	HSA?	ND	S	ND	Cu
OW5 -73	HSA?	ND	S & Gvl	ND	Cu
OW6 -73	HSA?	ND	S & Gvl	ND	Cu
OW7 -73	HSA?	ND	S?	ND	Cu
OW8 -73	HSA?	ND	S & Gvl	ND	Cu
OW9 -73	HSA?	ND	S & Gvl	ND	Cu
OW10-73	HSA?	ND	S?	ND	Cu
OW1 -74	SSA	ND	ND	ND	ND
OW2 -74	SSA	ND	ND	ND	ND
OW3 -74	HSA-MR	Ag S-5 & WS-5	ND	ND	Cu
OW1 -75	MR	G	Gvl	B	Cu
OW2 -75	MR	G	Gvl	B	Cu
OW1 -76	MR	ND	Gvl	B	Cu
OW2 -76	MR	ND	Gvl	B	Cu
OW1 -78	HSA	SS-8	Cu	B	Cu
OW2 -78	HSA	SS-10	ND	B	Cu
OW3 -78	HSA	SS-18	S & Cu	B	Cu
OW4 -78	HSA	SS-18	ND	B	Cu
OW5 -78	HSA	SS-19	Cu	ND	Cu
OW6 -78	JD	ND	Cu	B	Cu

Observation Well Data - Continuation

Boring	Boring Method	Sampling Method	Material in Bore Hole Annulus		
			Screened Interval	Seal	Backfill
TH4 -79	BH Ex	ND	NS	ND	NS
TH5 -79	BH Ex	ND	NS	ND	NS
TH6 -79	BH Ex	ND	NS	ND	NS
TH7 -79	BH Ex	ND	NS	ND	NS
TH8 -79	BH Ex	ND	NS	ND	NS
TH9 -79	BH Ex	ND	NS	ND	NS
TH10-79	BH Ex	ND	NS	ND	NS
TH11-79	BH Ex	ND	NS	ND	NS
TH12-79	BH Ex	ND	NS	ND	NS
TH13-79	BH Ex	ND	NS	ND	NS
TH14-79	BH Ex	ND	NS	ND	NS
TH15-79	BH Ex	ND	NS	ND	NS
OW1 -80	HSA	SS-6	NS	ND	S
OW2 -80	HSA	SS-12	S	B & Cmt	B & Cmt
OW3 -80	HSA	SS-6	S	B & Cmt	B & Cmt
OW4 -80	HSA & R	SS-11	S	B & Cmt	B & Cmt
OW1 -81	MR	ND	ND	ND	ND
OW2 -81	HSA	SS-8?	S	B & Cmt	B & Cmt
LW1 -81	AR	ND	S	ND	S
LW3 -81	AR	ND	Ws	ND	S
LW4 -81	AR	ND	Ws	ND	S
LW5 -81	AR	ND	Ws	ND	S
LW6 -81	AR	ND	Ws	ND	S
LW7 -81	AR	ND	S	ND	S
LW8 -81	HSA	ND	Wst & Gvl	ND	Gvl
LW9 -81	HSA	ND	Wst & Gvl	ND	Gvl
LW9A-81	HSA		caved	ND	Gvl
LW10-81	HSA	ND	Gvl	ND	Gvl
LW11-81	HSA	ND	E & Ws	ND	Gvl
OW1 -82	HSA	SS-28	S	ND	S
OW2 -82	HSA	SS-15	S	ND	S
OW3 -82	HSA	SS-35	S	B	NS

<u>LEGEND</u>

Boring: Number of Well

Boring Method: SSA   - Solid Stem Auger
                       HSA   - Hollow Stem Auger
                       MR   - Mud Rotary
                       BH Ex - Backhoe Excavation
                       AR   - Air Rotary
                       JD   - Jet Drilling
                       WB   - Wash boring
                       R   - Rotary - No mud used

Sampling Method: SS-2   - Split Spoon (number of samples indicated)
                       ND   - Not Designated or not collected
                       G   - Grab
                       Ws   - Wash samples
                       Ag S   - Auger Soil samples (Auger pulled with minimum rotation and material at base examined)

Material Screened Interval:
                       S   - Sand
                       ND   - None Designated
                       Gvl   - Gravel
                       NS   - Native Soil
                       WS   - Waste

Bore Hole Seal Above Screen:
                       ND   - None Designated or none installed
                       B   - Bentonite
                       Cmt   - Grout or cement

Backfill: S   - Sand
                       ND   - None Designated
                       Cu   - Cuttings
                       Gvl   Gravels
                       B   - Bentonite
                       Cmt   - Grout or cement
                       NS   - Native soil

The reliability of the well log data can be improved substantially
by field checking the locations, elevations, depths and static water
levels. Some of this work was done for wells in the vicinity of the
site by consultants (5,p.5) and other work, particularly in the
Musselman Lake and Ballantrae areas, by the Central Region of the
Ministry. However, excepting elevation data on private wells in the
immediate vicinity of the site, and for the onsite observation
wells, Figure 6 was based on data presented on the well logs.

To construct a regional map (16,v.1,map 2) the Applicant selected
only the logs of wells from the Ministry's files that penetrated 20
feet into the Main Aquifer (28,v.6,p.878). The construction
procedure used in our assessment considered all of the wells in
these files. These wells were subdivided into three groups,
according to depth. The regional map (Figure 6) is based on data
from wells in the two deeper groups.

Although Figure 6 was compiled independently, without reference to
other regional hydrogeologic maps, it presents much the same
configuration of ground-water flow as other regional maps (6,
Fig.A-3, and 8, Dwg.B-74522). This provides some confidence that
all of these maps present a reasonable picture of regional
ground-water flow.

APPENDIX   C

UNDERFLOW AND DILUTION IN THE MAIN AQUIFER

APPENDIX C

UNDERFLOW AND DILUTION IN THE MAIN AQUIFER

Calculations by Applicant

Underflow - Three values are presented by the Applicant for the quantity of water flowing beneath the site in the Main Aquifer. This water is assumed by the Applicant to be available for the dilution of contaminants that enter the Main Aquifer. The three values are:

1. 196.9 acre-ft/y or 101 igpm (7.83 L/s) (19,p.19). This is obtained by a water-balance method that calculates the amount of recharge to the Main Aquifer upgradient from the site.

2. 209 acre-ft/y or 108 igpm (8.18 L/s) (28,v.22,p.3797). This is obtained by using Darcy's Law to calculate the flow through a 15 foot thick cross-section of the Main Aquifer.

3. 252 acre-ft/y or 130 igpm (9.85 L/s) (39, p.4). This is presented as the ground-water flow rate beneath the whole of the site and is attributed to Reference 19 by the Applicant.

There is also a calculation that shows that the underflow value of 196.9 acre-ft/yr or 101 igpm (7.83 L/s) (1. above) requires an aquifer thickness of 3.4 m (11 ft) (19, p.24). The well log of OW1-81 showing an 3.4 m (11 ft) aquifer is included by the Applicant as corroboration that this flow is reasonable.

Dilution - The dilution factor for fluids entering the Main Aquifer from the landfill was calculated to be 4.7 (19,p.22). This value was later corrected to 5.7 (28,v.20,p.3506). The dilution factor was obtained by considering the amount of leachate generation onsite 41.7 acre-ft/y or 21 igpm (1.59 L/s) and the amount of underflow using 196.9 acre feet per year (1. above). The total ground-water

flow out of the site would then be 238.6 acre-ft/y or 122.8 igpm
(9.30 L/s).

Comments - There are a number of comments specifically related to
the above calculations.

1. The differences between the various estimates that are listed
    above are not considered to be significant to the assessment of
    the site. Some of the Applicant's calculations were probably
    made to test an hypothesis regarding a particular hydrogeologic
    system rather than to obtain a rigorous value for an aquifer
    parameter. These estimates are listed here to clarify
    discussions of the various submissions.

2. There appears to be an error in the Applicant's calculation
    (19,p.24) that an 3.4 m (11 ft) aquifer would be required to
    carry 196.9 acre feet of water per year through the Main Aquifer
    beneath the site. This calculation used an average ground water
    velocity which in turn was calculated from an aquifer
    permeability of 1.1×10^{-2} cm/s (2.8×10^2 igpd/ft^2)
    (16,p.19). The effective porosity of the aquifer was not
    considered in this calculation of velocity. If the velocity
    calculation is corrected to consider effective porosity then a
    12 m (40 foot) thick aquifer is required to transmit 196.9
    acre-ft/y or 130 igpm (985 L/s).

3. It could not be determined how the Applicant calculated the
    total underflow value of 252 acre-ft/y or 130 igpm (9.85 L/s)
    referred to in 3. above. However, it is very similar to the
    238.6 acre-ft/y or 122.8 igpm (9.30 L/s) value calculated by the
    water-balance method.

4. The presence of the ground-water mound in the north central part
    of the site would divert a portion of the upgradient dilution
    water moving towards the site from the north and east. Data are
    not available to quantify the effect of this diversion, but it
    would have some effect on the water-balance calculation.

5. The configuration of the water table in the Main Aquifer used by
    the Applicant and shown in his Progress Reports (11) is
    different from the configuration determined in this study
    (Figure 6). If this interpretation is correct, an area to the
    north of the site could act as the major source of ground-water
    recharge to the Main Aquifer beneath the site. This possibility
    is addressed in the next part of this report.

Independent Calculations

Figure 7 suggests that ground water approaches the site from the
north. As an exercise, the rate and source of this ground water was
investigated, assuming that this direction is correct and that
ground water is not moving from the Musselman Lake in the northeast,
as is suggested by the Applicant and by the regional map (Figure 6).

Consider flow through a one metre wide strip of aquifer at the north
boundary. The rate of flow can be estimated by Darcy's Law to be:

$$Q = KiA = 800 \ m^3/y$$

where $K = 1 \times 10^{-2}$ cm/s. This is considered to be a
          reasonable bulk value for the entire thickness of the
          aquifer.

where $i = 0.008$. This is the gradient between the 1035 and
          1030 foot contours on Figure 7 (5'/600')

where $A = 30 \ m^2$. This assumes a 30 m (100 feet) thick
          aquifer (Section 4.3.4.4).

Using a recharge rate of 0.3 m/y (12"/y), the length of the 1 metre
wide strip necessary to produce 800 m^3/y is roughly 2700 m (8859
ft.). This distance is reasonable considering that the distance to
the topographic divide north of the site is approximately 3500 m
(11484 ft).

Both this water balance and that of the Applicant assume that ground water mound in the northeastern portion of the site has a negligible influence on the ground water flow underneath the site. This may not be the case as was discussed earlier in this appendix.

In calculations such as this, it is always possible to juggle numbers; for example, increasing the aquifer thickness beyond 30 m can be accommodated by decreasing the bulk K of the aquifer. Such calculations merely indicate possibilities and test hypotheses.

The hydrogeologic regime (i.e. influent swamp and lake) that has been generally accepted is uncommon in a humid climate. It seems necessary to invoke a sensitive relationship between the water budget and the hydraulic conductivity of the bottom of the swamp and lake so that the exfiltration is sufficient to account for the underflow, but at the same time, does not cause the swamp and lake to go dry in the late summer. If this aspect of the site assessment is considered to be important, additional work could be done including the installation of piezometers to verify that gradients are vertically downward beneath the swampy lowland and Musselman Lake (i.e., show that these features are influent). At present, downward gradients are inferred from installations on the site outside of the margin of the swamp and from Figure 6.

General Comments

A number of general comments on dilution of contaminants are pertinent to an assessment of this site.

1) Dilution by underflow assumes that the contaminants, after reaching the Main Aquifer, completely mix with all of the underflow, rather than travelling as a discrete mass within the underflow. The extent of mixing that does occur depends on the amount of dispersion in the Main Aquifer. Dispersion is controlled by a number of factors including the distance travelled, the velocity of the ground water, the anisotropy of the aquifer and the presence of zones of relatively high

- 98 -

permeability in the aquifer. The dispersive nature of the aquifer has not been investigated by any workers.

2) Perhaps it would be appropriate to consider dilution within a thin aquifer 4.6 m (15 ft) to compensate for incomplete mixing throughout the total thickness of the aquifer. However, if this is done, it may be necessary to reexamine the quantity of dilution water determined by the water-balance method to consider this thinner zone in the Main Aquifer.

3) A major problem associated with the application of dilution calculations to this site is that the amount of dilution required to reduce the concentration of contaminants on the site to "acceptable" levels is not known.

Conclusions

The water-balance method of calculating underflow requires knowledge of the influence of the ground-water mound on site, the position of the ground-water divide to the north and the ground-water trough to the east of the site and the amount of recharge. The calculation using Darcy's Law requires knowledge of the thickness, gradient and the bulk hydraulic conductivity of the Main Aquifer. We have little confidence in our ability to quantify these factors and thus there is a large degree of uncertainty in the dilution calculations and the estimates of expected contaminant levels based on these calculations.

It would be more useful if direct measurements of the concentrations of conservative contaminants, moving on the same flowpath, downgradient in this plume could be used to assess dilution at this site rather than the indirect methods discussed in this Appendix. Therefore, it is felt that while the existing dilution calculations are of interest in assessing the level of the understanding of the hydrogeology of the site, they are of limited value in the assessment of the potential impact of this site on surrounding properties.

APPENDIX D

WATER WELL INFORMATION AND PUMPING TESTS

Water Well Information and Pumping Tests

Introduction

This Appendix presents a tabulation of the well information and an assessment of the testing of wells in the vicinity of the Stouffville Municipal Wells, the Stouffville Municipal Wells, two wells on the York Sanitation Landfill Site No.4, and two private wells located on the property of M. Wade Construction Co. Ltd., west of the Landfill Site. Most of this information has been used by various workers in assessments of the hydrogeologic characteristics of the Main Aquifer at the site and in the design of the purge well system proposed for the contingency plan.

Generally the pumping test data at the wells listed excepting the Municipal wells are deficient with respect to calculating aquifer characteristics, in particular hydraulic conductivity, transmissivity, storage coefficient, depth, and radius of influence. For example, the hydraulic conductivity of the aquifer could be calculated using only three of the municipal well tests. Therefore, for comparison purposes, specific capacities have been calculated for the wells and are presented in all three tables in this Appendix.

The specific capacity of a well is related to the transmissivity and storage coefficient of the aquifer according to Bentall (42,p.339) and Todd (43,p.158). Therefore the specific capacity can be used for general comparison purposes between wells. It should be stressed, however, that well efficiency plays a significant role in limiting the usefulness of using the specific capacity figures per se. Well efficiencies have not been evaluated in this review.

Wells in the Vicinity of the Municipal Wells

A compilation of nine wells drilled between 1956 and 1966 in the
vicinity of the present Stouffville Municipal Wells is presented in
Table 1. Based on the well records of those nine wells, it is
estimated that the aquifer (waterbearing unit) at the municipal well
site located in the Township of Whitchurch Stouffville, Conc. 8, Lot
8, could have a thickness of between 2-22 m (8 to 71 ft). The
lithology of this aquifer varies from "sand, to coarse sand, to
gravel". An overlying "confining" layer comprised of "clay, clay
sand and gravel, or clay and stones", ranges in thickness from 0 to
11 m (35 ft). The specific capacity has been estimated to be from
2.8×10^{-1} to 14 L/s/m dd (litres/second/meter drawdown) or 1.1 to
54 igpm/ft dd (imperial gallons/minute/foot drawdown).

Municipal Wells

Table 2 lists the results of the pumping tests conducted at
Municipal Well No.5, No.10, and the Well near Well No.8. In the
reports that were available from the Regional Municipality of York
and completed by Scope in 1967, (31, 32, 33) descriptions of samples
of the geologic materials in the aquifer are lacking. The aquifer
in Well No.10 is reported by Scope (33) to be 7.3 m (24 ft) thick
although the corresponding well record 8210 shows a thickness of
about 10 m (34 ft). Aquifer thickness at the other two wells is
based on cross-sections presented by Hydrology Consultants (8,
Fig.A-2) and well record D8212 and 8211. The aquifer is estimated
to be 10 m (32 ft) thick at Stouffville Well No.5 and the 19 m (62
ft) thick at the Well No.6 near Test Well No.8.

The aquifer characteristics, based on the test data, indicate that
generally the aquifer is under water-table conditions during
pumping, has a transmissivity in the range of 200 to 685 m^2/day,
(13,400 to 46,000 igpd/ft), and has corresponding hydraulic
conductivity values in the range of 1×10^{-2} to 6×10^{-2} cm/s
(3×10^2 to 2×10^3 igpd/ft^2).

TABLE 1.   WELL RECORD SUMMARY - TOWNSHIP OF WHITCHURCH-STOUFFVILLE, CONCESSION VIII, LOT 8

Well No.	Date	Driller	Water Bearing Zone ft.	Material Thickness	Test Rate igpm	Static Level ft.	Pumping Level ft/hrs	Specific Capacity igpm/ft.d.d.
8213	Feb., 1956	Boadway	16-32	16	60	4	10/2	10
8214	Feb., 1956	Boadway	18-42	24	60	9	15/2	10
8215	Feb., 1956	Boadway	15-39	24	60	5	10/2	12
8216	Mar., 1956	Boadway	18-40	22	60	7	12/2	12
8212	Mar., 1960	Rutledge	18-50	32	700	9.5	22.6/48	54
8208	Apr., 1965	Faulkner	15-85	70	100	8	65/1	1.7
8209	Apr., 1965	Faulkner	1-60	60	75	18	50/2	2.3
8210	Aug., 1965	Faulkner	63-71	8	50	5	50/58	1.1
8211	Jan., 1966	Faulkner	7-70	63	351	8	59/48	5.9

Note: Data are insufficient to calculate Transmissivity or Hydraulic Conductivity of Main Aquifer in this area.
        Conversion Factors ft. x .3048 = m
                          igpm x .0758 = LPS
                          igpm/ft.d.d. x 2.49 x 10^4 = LPS/md.d.

TABLE 2:   SUMMARY OF PUMPING TESTS - TOWN OF STOUFFVILLE MUNICIPAL WELLS
(See Notes* for explanations of terms)

WELL REF. NO.	PW #5	TW #10	PW #6 @ TW #8
REPORT REFERENCE	32, June, 1965	33, Aug.1965	31, Dec. 1965
TRANSMISSIVITY	40,300 (d/dd-SL)	180,000 (t/dd-8 SL)	17,700 (t/dd-10 SL)
igpd/ft	46,000 (t/dd-3 LL)	14,300 (t/dd-8 LL)	13,400 (t/dd-LL)
HYDRAULIC CONDUCTIVITY			
igpd/ft^2	1,200 to 1,400	420 to 600	216 to 285
cm/s	5×10^{-2} to 6×10^{-2}	2×10^{-2}	8×10^{-3} to 1×10^{-2}
STORAGE COEFFICIENT	2.8×10^{-2} (d/dd-SL)	1.0×10^{-2} (t/dd-8 SL)	4×10^{-4} (t/dd-10 SL)
S	1.6×10^{-2} (t/dd-3 LL)	4×10^{-4} (t/dd-8 LL)	5×10^{-4} (t/dd-LL)
RADIUS OF ZERO DRAWDOWN	660 @ 1 (d/dd-SL)	1,200 ft. @ 0.5	1,600 @ 1.4 (d/dd-SL)
ft @ time in days		(hypothetical d/dd-SL)	6,000' @ infinity
TEST YIELD AND DURATION	466/24	50/48	350/48 hr.
igpm/hrs			
SPECIFIC CAPACITY	45	1.1	5.9
igpm/ft dd			
OBSERVATION WELLS	3, 4, 8 9	8 and 9	8, 9 10
WELL RECORD NO.	8212	8210	8211
FORMATION THICKNESS	32	34-24	62
ft			

*NOTES: i) Underlined numbers are those which appear in the reports
            referenced. Non-underlined numbers were obtained by the
            re-evaluation of test data by this review. Where only an
            underlined number is given, this indicates agreement with the
            number presented in the reports.

       ii) Aquifer characteristics in the reports were calculated using the
            Jacob semilog method. Prickett non-steady state water-table
            log-log type curves were used in our re-evaluation of the
            original data for comparison.

      iii) d/dd - SL means distance/drawdown graph semilog
            t/dd - 8 SL means time/drawdown graph semilog graph of well No.8
            t/dd - 8 LL means time/drawdown graph log-log graph of well No.8

       iv) Numbers presented in this table are considered to be reasonable
            and reliable based on the original data and indicate a variation
            of aquifer characteristics at the Municipal Well Site.

International Water Supply Ltd. (8,p.3) suggested that the aquifer transmissivity "appears to be in the order of 20,000 to 40,000 igpd/ft. (298-596 m^2/d). These figures appear to be reasonable and within the range of values listed on Table 2 and indicate the degree of heterogeneity in local aquifer conditions.

Wells on the Landfill Site and Wade Wells

Table 3 presents the results of the pumping tests on two wells at the landfill site and, from two wells on the Wade property. The Wade irrigation well (15835) is located approximately 800 m (2600 ft) west of the landfill. The Wade house well (11705) is located approximately 1700 m (5700 ft) west of Highway 48 adjacent to the site. The aquifer on the Wade property is up to 27.43 m (90 ft), thick (Well Record No. 15835) with the lower confining zone at an elevation of about 263 m (875 ft) AMSL. In comparison the well log of Observation Well 1-81 the most westerly well on site, shows only 3.4 m (11 ft) of aquifer material above a confining layer. Well 1-81 may not actually penetrate the entire aquifer thickness as discussed in Section 4.3.4.4. The Wade house well and the York Sanitation well did not penetrate to a confining layer. The specific capacities of the four wells range from 1.44 to 4.35 igpm/ft dd (3.6 x 10^{-1} to 1.1 L/s/m dd).

Conclusions

The specific capacity of the wells tested is shown to vary from 54 to 1.1 igpm/ft dd (14 to 2.8 x 10^{-1} L/s/m dd). The average of 16 calculated values is approximately 10 igpm/ft dd (2.5 L/s/m dd) equivalent to a transmissivity of up to 20,000 igpd/ft dd (300 m^2/d). The wide range in values probably reflects both aquifer variability and the accuracy of the testing methods used. The specific capacity values also appear to be quite variable and random in aerial distribution.

The hydrogeologic characteristics of the Main Aquifer at the York Sanitation Landfill Site No.4, appear similar to those of the

TABLE 3.   <u>SUMMARY OF PUMPING TESTS - SITE AND WADE WELLS</u>

Well	York Sanitation #1 #9450		OW1-81	Wade/House #11705	Wade/Irrigation #15835
Year	1969		1981	1973	1981
Driller	Gormly		?	Boadway	Boadway
Depth/Diam.	154 ft/7 in		210 ft/5 in	185 ft/5 in	238 ft/6 in
Water Found (elevation)	131 ft (1014 ft)		149 ft (970 ft)	975 ft (1003 ft)	145 ft & 235 ft (965 ft)
Formation	m Sand		Sand & Gravel	Sand & Gravel	Layered Sand & Gravel
Thickness Penetrated	24 ft		11 ft	10 ft	90 ft.
Screen/Diam.	8 ft/7 in		15 ft/2 in	8 ft/5 in	44 ft/6 in
Slot	?		?	20 & 25	30, 16, & 40
Setting	146-154 ft		149-162 ft	170-178 ft	187-231
<u>Pumping Tests</u>	<u>1969</u>	<u>1980</u>	<u>1981</u>	<u>1973</u>	<u>1981</u>
Yield	36 igpm	10 igpm	2.2 igpm	12 igpm	270 igpm
Duration	3 hrs	3 hrs	8 min	2.5 hrs	12 hrs
Drawdown	21 ft	6.92 ft	9.25 ft	3 ft	62 ft
Pumping Level	151 ft	126.5 ft	94 ft	100 ft	164 ft
Static Water Level	130 ft	120 ft	104 ft	97 ft	102 ft
Specific Capacity igpm/ft	1.7	1.4	1.6	4.0	4.4

Data available is in sufficient to calculate Transmissivity or Hydraulic Conductivity of
Main Aquifer in this area.

aquifer at the Stouffville Municipal Wells. However, until complete
data at the site are available from comprehensive pumping tests the
data on the aquifer characteristics at the site must be considered
as approximations and of insufficient accuracy to incorporate as
actual values in, for example, a final purge well design.

APPENDIX E

THE VERTICAL HYDRAULIC GRADIENT IN
THE MAIN AQUIFER BENEATH THE SITE

APPENDIX E

THE VERTICAL HYDRAULIC GRADIENT IN
THE MAIN AQUIFER BENEATH THE SITE

Four pairs of wells can be used to determine the vertical gradient
in the Main Aquifer beneath the site. The four well pairs are
OW1-75 and OW2-75, OW1-76 and OW2-76, OW2-80 and OW3-80, and OW3-80
and OW4-80. The wells are located in the southwestern and
northeastern portions of the site and are shown on Figure 7.

The following table lists the values for the head loss (h), the
distance over which the head loss occurs(L), the vertical gradient
(^{i}v) and the vertical direction of ground-water flow between each
well pair.

VERTICAL HYDRAULIC GRADIENTS IN MAIN AQUIFER ON SITE

Well Nest	h (feet)	L* (feet)	^i_v	Vertical Direction of Ground-Water Flow
OW1-75 OW2-75	0.16	18.00	0.009	down
OW1-76 OW2-76	0.17	40.50	0.004	up**
OW2-80 OW3-80	1.35	22.50	0.06	down
OW3-80 OW4-80	0.42	60.75	0.007	down

*   see attached well logs for specifics
**   anomalous direction. Elevations should be checked if value of
     gradient becomes important and direction remains anomalous.

The h values used for both the OW1-75, OW2-75 and OW1-76, OW2-76
well pairs are the averages for the 1976-1981 measurement period.

The h values for both the OW2-80, OW3-80 and OW3-80, OW4-80 well pairs are the averages for the period from May 1981 to February 1982. Earlier data were not used because of the relative instability of the water levels in OW3-80 and OW4-80. Continued water-level data for these two wells would be necessary to test the validity of the calculated i_v values and the downward direction of vertical flow.

The L values were determined by considering both that the head in gravel or sand pack is that of the most transmissive unit exposed to the gravel or sand pack and that the head loss through the gravel or sand pack is negligible. The L values used to calculate the gradients are shown on the accompanying well logs.

The i_v values represent the overall vertical gradient between the gravel or sand packs. The actual i_v values through individual geologic units would depend on the relative thicknesses and vertical hydraulic conductivities of the units and would not therefore, be equivalent to the i_v values listed in the above table.

Project Name:	Site #4		Borehole No:	OW1-75 & OW2-75
Job No.	976-184		Date Completed:	Aug. 1975
Client:	York Sanitation		Geologist/Engineer:	
Borehole Type:	6"ø Mud Rotary		Elevation:	1127.24
Location:	Lot 14 Concession 8			

Profile

Stratigraphy

Elevation (feet)

Description & Remarks

OW1-75

OW2-75

Simple

Number

Type

Blows/Foot

Penetration Test
Blows / Foot

20 40 60 80

Piezometer or
Standpipe
Installation

OW1-75

medium gravel
medium to fine
sand

coarse gravel,
sand

medium to fine
sand, gravel

coarse to medium
gravel, sand

clay sand till

OW2-75

Sand, medium

to fine gravel

OW2-75
Gravel pack

OW1-75
Gravel pack

Elevations marked: 1140, 1130, 1120, 1010, 1000, 990, 980

Piezometer elevations: 1020, 1010, 1000, 997, 992, 987

ΔL = 18

APPENDIX E

Modified from
Conestoga Rovers & Associates
Progress Report May 1980

Project Name: Site #4
Job No. 976-184
Client: York Sanitation
Borehole Type: 5" ⌀ Mud Rotary
Location: Lot 14   Concession 8
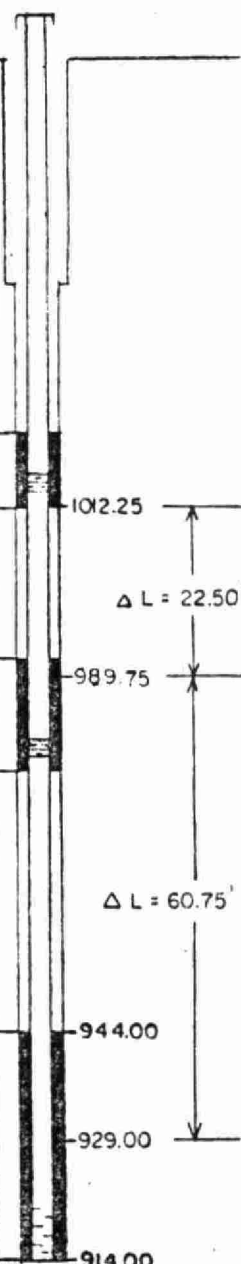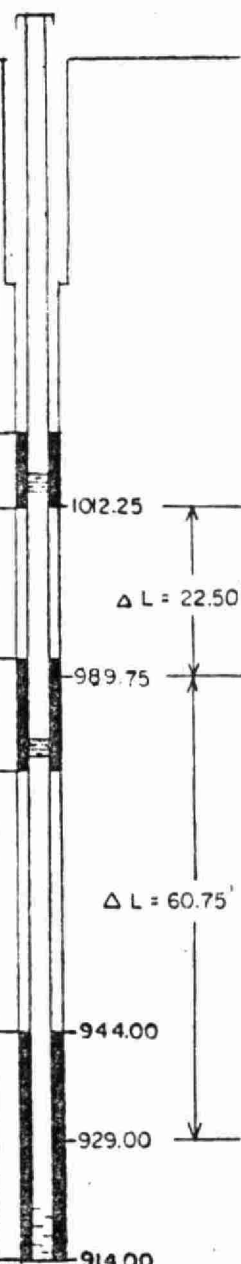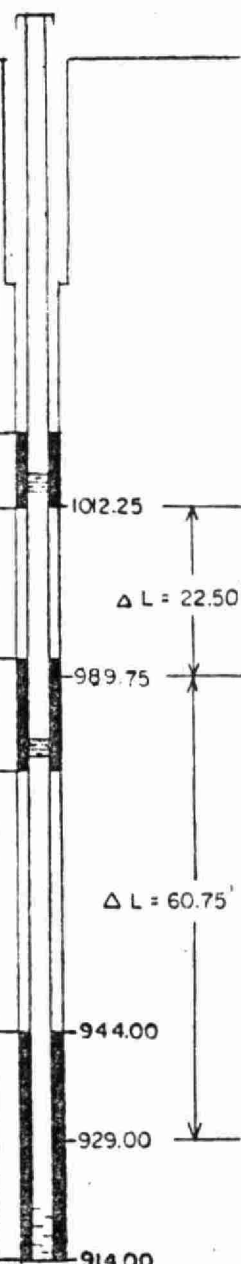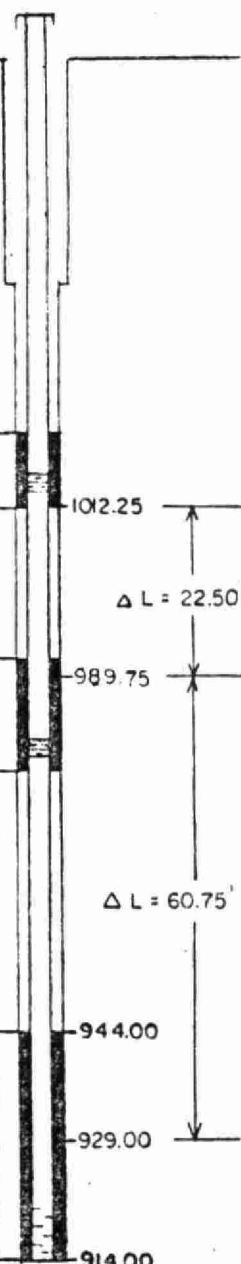
Borehole No: OW1-76 & OW2-76
Date Completed: Jan. 1976
Geologist/Engineer:_____
Elevation: 1126.5

Elevation (feet)	Stratigraphy	Profile — Description & Remarks OW1-76 and OW2-76	Simple — Number	Type	Blows/Foot	Penetration Test Blows/Foot 20 40 60 80	Piezometer or Standpipe Installation
1030		sand and gravel					
1020		no sample					1019
						OW2-76 gravel pack	1014
1010		coarse sand & pebbles					1009
		sand and gravel					
1000		gravel and sand					ΔL = 40
990		fine medium sand					
980							980
		gravel, sand, white silt				OW1-76 gravel pack	973.5
970							967
960		grey clay, gravel					

APPENDIX E

Modified from
Conestoga Rovers & Associates
Progress Report May 1980

Project Name: Site No. 4
Job No. 976-184
Client: York Sanitation
Borehole Type: 3" 0 Casing Rotated & Jetted
Location: Lot 15, Concession 8

Borehole No. OW2-80, OW3-80, OW4-80
Date Completed July 9, 1980
Geologist/Engineer A.V.N.
Elevation 1076.14 ft. AMSL, top of casing

Elevation (feet)	Stratigraphy	Profile — Description & Remarks	Number	Type	Blows/Foot	Penetration Test Blows/Foot 20 40 60 80	Piezometer or Standpipe Installation
0 (1074)		Brown silty sand (SM)					
		Grey clay (CL)					
		Brown silty sandy till (SM)					
		Brown silty clay till (CL)					
30 (1044)		Grey silty clay till (CL)					
50 (1024)		Grey clay (CL)					
60 (1014)		Grey silty fine sand very wet and flowing (SM)				Sand Pack OW2-80	1012.25
70 (1004)		Grey fine sand (SP)					ΔL = 22.50'
80 (994)		Grey medium sand (SP)					989.75
		Grey silty fine sand (SM)				Sand Pack	
90 (984)		Grey silt (ML)				OW3-80	
95 100 (974)		Grey silt, trace of fine sand (ML)					
		Grey silty, fine sand (SM)					
110 (964)		Thin gravel layers in silt (ML)					ΔL = 60.75'
120 (954)		Grey sandy silt (SM)					
		Grey fine sand with some gravel (SP)					
130 (944)		Grey silty fine sand with occasional gravel size pieces (SM)					944.00
140 (934)						Sand Pack	929.00
150 (924)						OW4-80	
160 (914)							914.00

Modified from
Conestoga - Rovers & Associates
Progress Report May 1980